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MODELING THE DYNAMIC RESPONSE OF STRUCTURES TO SUPPORT OFFSHORE WIND TURBINES

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CONTENTS

SYNOPSIS

In this thesis, it was the goal to design, analyze and subsequently perform an optimization of the structure of a monopile-type offshore wind turbine.

First, it was necessary to study and subsequently analyze the theories that would allow us to carry out the thesis. This consists in finding a theory that allows modeling of the soil and another that allows the application of the forces corresponding to the wind and the sea. Soil modeling theory considers the soil as a distributed spring model, which according to the literature it is considered a more reliable representation of the soil. As for the theory of force application for the water and wind, the worst possible environmental conditions are considered to carry out the last load test. The last load test is to analyze the structure for the worst situation that could occur, and therefore this represents the worst conditions of wind and sea together, at the same time. It is important to note that modal analysis techniques are used to calculate the eigen frequencies and modes of the structure. This analysis is very important in the design, because it will mark the dynamic behavior of the structure.

Second, once the literature discussing the different theories to be taken into account is well examined, first design of the structure is modeled and realized by taking into account a benchmarks case discussed in the literature. A remark should be made that for this first design some parameters defined in another thesis are considered, such as the characteristics of the turbine, because it seeks to compare the results obtained with this thesis. In this way we analyze and compare the results obtained, as shown in the thesis can be seen these results as good, because they achieve the same response values during the last load test and modal analysis from the discussed reference, but with a much simpler design .

Following this analysis, we performed an optimization of the initial design, but we tried to achieve a better response in all analyzes resulting in a less expensive design. This process takes place successfully because considering that the first design could be regarded as good for the values obtained, these values are improved response in around 10% in all respects. Finally we can consider that the goal of this thesis achieved by optimizing the design of the turbine foundation.

LIST OF SYMBOLS

a, $\ddot{x}$	Acceleration of the mass	[N/s^2]
α	Thermal coefficient	[C^{-1}]
d_w	Water depth	[m]
f_n	Natural frequency	[Hz]
g	Acceleration of gravity	[N/s^2]
g_k	Design variables	[-]
h_k	Design variables	[-]
k	Spring stiffness	[N/m]
m	Mass	[Kg]
n	Number of springs	[-]
n_i	Sequence of digits	[-]
t	Time	[-]
v_1	Wind speed	[m/s]
x	Desplacement	[m]
A	Area	[m^2]
C_D	Drag coefficient	[-]
C_M	Inertia coefficient	[-]
$C_T(\lambda)$	Trust coefficient	[-]
D_o	Monopile`s outer diameter	[m]
Es	Young’s modulus	[Pa]
F_D	Drag force	[N]
F_M	Inertia force	[N]
Fs	Force applied in the mass	[N]
F_T	Trust force	[N]
$F(x)$	Objetive function	[-]
$F'(x)$	Derivate function	[-]

$F''(x)$	Second derivate function	[-]
Gs	Shear modulus	[Pa]
H, F_H	Horizontal load	[N]
Hs	Wave height	[m]
$H_k(i)$	Hammersley points	[-]
K	Soil stiffness	[N/m^3]
L	Wave length	[m]
R	Rotor radius	[m]
$2R$	Tower diameter	[m]
T	Wave period	[s]
U	Velocity of flow normal	[m/s]
$\ddot{U}$	Acceleration of the normal flow	[m/s^2]
Vr	Maximum rotor speed	[rpm]
V_{min}	Minimum rotor speed	[rpm]
X_L	Lower limit	[m]
X_U	Upper limit	[m]
ρ_a	Air density	[Kg/m^3]
λ	Tip speed ratio	[-]
Ω	Rotor speed	[rad/s]
ρ_w	Water density	[Kg/m^3]
ω	Angular wave frequency	[Hz]
Ψ	Wave number	[-]
υ	Poisson's coefficient	[-]
1P	Rotation frequency of the turbine	[Hz]
3P	Blade passing frequency of three-bladed turbine	[Hz]

FIGURES

TABLES

1. THEORETICAL FRAMEWORK

1.1 INTRODUCTION

The growing requirement for clean and sustainable energy production in the near future has resulted in the search for alternatives to fossil fuels as an energy source. As a result of that, wind energy is one of the most promising options for generating electricity

A constant search for greater wind potential has pushed the industry from onshore towards to offshore solutions with superior wind conditions. Aiming for more effective wind conditions corresponds to seeking for more remote offshore sites and consequently higher sea depths. Installing the wind turbines at such depths involves high stakes and high expenses, both from the financial and the engineering point of view.

Nonetheless, several different foundation structures for various sea depths and soil conditions have been proposed for the offshore wind turbines. Thus one of the major problems encountered in relation to offshore wind turbine foundations is the connection of the structure to the ground and in particular how the loads applied to the structure should safely be transferred to the surrounding soil. Furthermore, both offshore wind turbines and their foundation structures must be more reliable than onshore due to higher service and repair costs at such sites [7].

We can see the Figure 1 the most common types of foundations, which are explained briefly below:

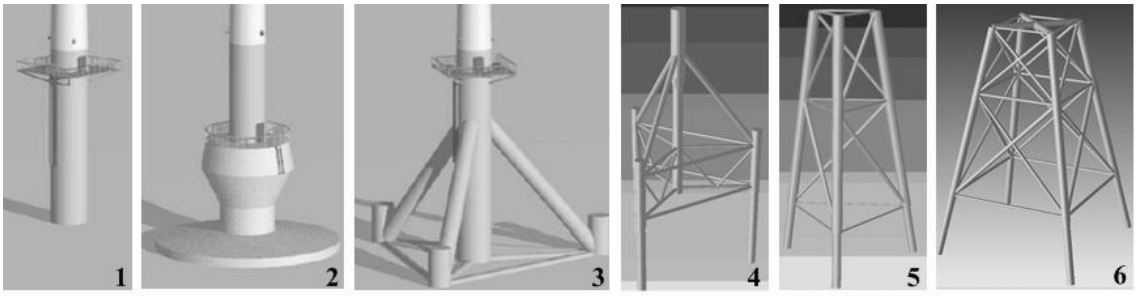

Figure 1: Offshore wind turbine support structures [7]

Structure 1 – Monopile Foundation

This is a simple structure consisting of a steel pipe piled into the seabed by driving and/or drilling methods.

A larger diameter sleeve is attached to the pile by concrete casting, where its top rim is a flange that accommodates fixation of the turbine tower by bolting.

Structure 2 – Gravity Based Foundation

This structure is currently used on most offshore wind projects at shallow water depths up to 5 *m*. It consists of a large base constructed from steel and concrete, resting on the seabed. It relies on weight of the structure to resist overturning; hence the turbine is dependent on gravity to remain erect. The structure is resistant to scour and deformation due to its massive weight. The wind turbine tower is attached similarly to monopole foundations.

Structure 3 – Tripod Foundation

This design is typically used for platforms in the oil and gas industry. It is made from steel tubes welded together, typically 1 to 2.5 *m* in diameter. It is anchored 20 to 40 *m* into the seabed by means of driven or drilled piles from 1 to 2.5 *m* in diameter. The transition piece is typically attached onto the centre column by means of concrete casting as well.

Structures 4, 5, 6 – Jacket Foundations

Jacket structures are made from steel tubes, typically 0.5 – 1.5 *m* in diameter, welded together to from a structure similar to lattice towers. They are anchored to the seabed by driven or drilled piles, ranging from 1 – 2.5 *m* in diameter. Several 3 to 4 legged jacket structures have been proposed as illustrated in Figure 1(2).

As can be seen the choice of either foundation depends on a number of contour conditions such as [7]:

- Water depth and soil conditions to determine the appropriate foundation structures, as well as if reinforcement is needed

- Turbine loads and wave loads to determine the overall size of the structure and the extent of reinforcement
- Manufacturing and installation demands based on statutory requirements and expected lifetime.

Consequently, the choice of the foundation design has to be considered from many aspects, but both the public opinion and investment costs agree with simple and discrete foundation structures. It is conspicuous that the monopile foundation structure in Figure 1(2) appears quite simple compared to the other designs. This is also the reason for its popularity in the majority of projects located at water depths up to 30 m. The certification company DNV covers the technical documentation for such structures with a set of design rules given in the standard [DNV-OS-J101, 2007] – Design of Offshore Wind Turbine Structures [7].

Detailed elaboration of a monopile foundation and the overall design criteria according to this standard are given in section 3.

1.2 OBJECTIVE

The main objective of this project will be to study, design and optimization of a wind turbine monopile type, based on the analysis of the last load test.

To do this I will perform a research, study and definition of the boundary conditions under which the wind turbine will be submitted in the last load test, that will be necessary to establish the most unfavorable situation that defines this essay, but also theories that allow us to perform the analysis and implementation of all these conditions on our wind turbine.

We should also mention that to perform this work I will compare the results with those of another thesis which has designed a jacket wind turbine by the same boundary conditions.

I will focus on the study and design of wind turbine monopile type because it’s one of the most common types of foundations and because of that one it has more future. Also I will try to perform an optimization of the design made it possible to reduce costs and better definition of the dimensions to get the best possible performance.

With this thesis I also try to acquire a battery of knowledge about wind energy and also about ANSYS finite element program, with which it will perform the design, testing and optimization of wind turbine.

1.3 MONOPILE FOUNDATION AND STRUCTURE

Monopile foundations have been used for offshore oil and gas platform foundation for decades. In this context, they are known as pile-sleeve connections. A pile-sleeve connection consists of a sleeve mounted concentrically on a pile that is driven into the seabed, with the larger diameter sleeve placed around the smaller diameter pile forming annuli between them.

The sleeve related to wind turbines is also known as transition piece, as it joins the wind turbine tower to the pile. An illustration of the connection concept, as well as image of an installed structure is shown in Figure 2.

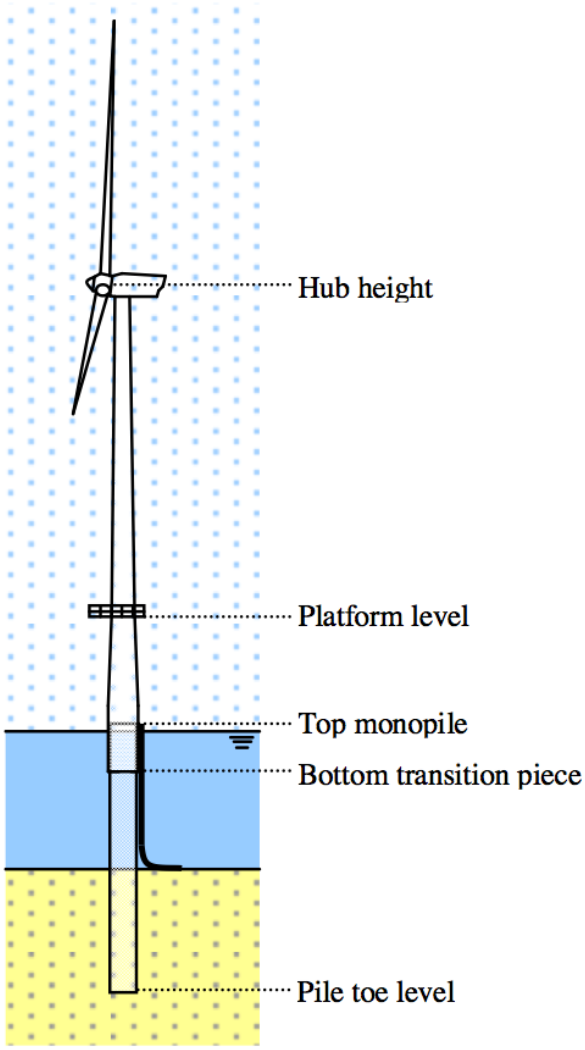

Figure 2: Design levels for an offshore wind turbine [5]

The pre-fabricated transition piece in Figure 2 is usually embracing the pile, although the opposite is possible, but impractical for mounting external equipment such as ladder and cables. As can be seen in [7], the dimensions of this type of structures usually have variation within the following ranges.

- Appropriate for water depths: 5 – 30 *m*
- Outer pile diameter, *Dp*: 3 – 6 *m*
- Thicknesses of the pile and transition piece, *tp* and *ts*: 40 – 80 *mm*
- Penetration into the soil: 20 – 40 *m*, depending on the soil
- Tower length: 60-80 m

These variations we consider are due to different contour conditions we find in each place and also the type of turbine that we will use.

We remark that in many cases the connection between the transition piece and the monopile is performed by high-strength concrete, but in our case we will try to simplify this and do it all with the same material to simplify the construction and design.

Based on the data in [1] we will consider the following stratification of the seabed for our analysis.

Depth (m)		**Material**
From	**To**	
0,0	5,0-6,7	Very soft to soft clay
5,0-6,7	7,0-9,5	Loose silty sand to silty sand
7,0-9,5	10,0-10,7	Medium dense sand and stiff sandy clay
10,0-10,7	11,1-15,0	Sand and gravel alternating with gravel and cobbles
11.1-15,0	17,0	Moderately weathered rock to completely weathered rock
17,0	28	Rock

Table 1: Seabed stratification [1]

NREL 5 MW wind turbine is considered to perform the design and the optimization. Specifications are detailed in chapter 2.

1.4 THEORY IS BASED ON OUR PROJECT

For make the last load test it is needed to define the boundary conditions and also the theories that allow us to implement these conditions.

These boundary conditions are defined depending on the state of wind, water conditions, soil stratification and interaction monopole-soil considered.

So we need theories that allow us to apply these conditions to our wind turbine for the analysis of the force exerted by water, wind resistance and will exercise the ground, so that's what we do below.

1.4.1 Theoretical basis of the boundary conditions

1.4.1.1 Wind

The wind velocity can be considered useful to harness energy if it is above 3 m/s (light wind), but full production (though varies with device) requires 12 m/s (strong wind). The wind to stop electricity generation is above 25 m/s (storm). The aerodynamic force generated by the wind on a turbine can be assumed proportional to the wind dynamic pressure $v_1^2\rho_a/2$ multiplied by the rotor swept area πR^2, where v_1 is the far upstream wind speed, ρ_a is the air density, and R is the rotor radius. Then the thrust force is giving by [6]:

$$F_T = \frac{1}{2}\rho_a \pi R^2 v_1^2 c_T(\lambda) \qquad (1)$$

Where the thrust coefficient C_T accounts for the fact that the blades are rotating, therefore, it is a function of the tip speed ratio, $\lambda = \Omega R/v_1$, where Ω is the rotor speed in rad/s.

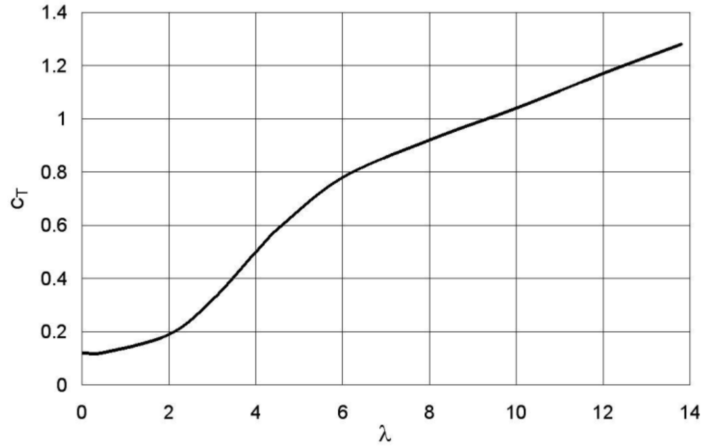

Figure 3: Thrust coefficient of a turbine as a function of the speed tip ratio [6]

1.4.1.2 Water

Waves induce vortices of water particles, which generate drag forces on obstacles. In addition, a fluid moving horizontally also generates pressures over obstacles. If a dominant extreme wave is idealized then hydrodynamic loads can be obtained from the drag and inertia forces applied on a submerged turbine tower as follows [6]:

$$F_D = \frac{C_D \rho_w (2R) H_s^2 \omega^2}{16\psi} \frac{\sinh(2\psi d_w) + 2\psi d_w}{\cosh(2\psi d_w) - 1} \qquad \textbf{(2)}$$

$$F_M = \frac{\pi C_M \rho_w (2R)^2 H_s \omega^2}{8\psi} \qquad \textbf{(3)}$$

where $C_D \approx 0.7$ and $C_M \approx 2$ are empirical coefficients for drag and inertia for smooth tubular sections, ρ_w is the water density, $2R$ is the tower diameter, Hs is the significant wave height, d_w is the water depth, $\omega = 2\pi / T$ is the angular wave frequency and T is the wave period, and $\psi = 2\pi / L$ is the `wave number' with L being the wave length. The wave number can be obtained from:

$$\omega^2 = g\psi \tanh(\psi\, d_w) \qquad \textbf{(4)}$$

in a deep water case $\omega^2 = g\psi$, where g is the acceleration of gravity. When the drag force and the inertia force per unit length are integrated from the seabed to the water surface, the former varies with time through a $\cos^2$ function whilst the latter varies with time through a sin function.

Therefore, the total horizontal load H can be expressed by:

$$H = max\{F_D \cos^2(-\omega t) + F_M \sin(-\omega t)\} \qquad \text{for } -\frac{T}{4} \leq t \leq 0 \qquad (5)$$

Apart from the horizontal load component, waves can induce an important vertical cyclic load component, pull and push during trough and crest respectively. This vertical cyclic loading is important in terms of displacements and stiffness rather than in terms of foundation resistance.

1.4.1.3 Soil

Characteristic loads for the design of wind turbines in ultimate limit states are generally established by employing time-domain aeroelastic response simulations. The accuracy of the derived loads depends on the number of simulations and on how realistically the models used to represent the turbine, support structure, and foundation describes the true structural response. One potential shortcoming in modeling foundations relates to their flexibility.

A single pile (often referred to as a monopile) is the most common type of foundation used today for offshore wind turbines; the support structure connects to such a pile foundation that extends some depth below the mudline.

It will be considered distributed springs (DS) model [11] because it includes the true length of the monopile foundation and replaces layers of the soil with linear elastic springs and it represents a real soil modeling, so the distributed springs (DS) model replaces the soil surrounding the pile foundation with springs distributed along the length of the pile.

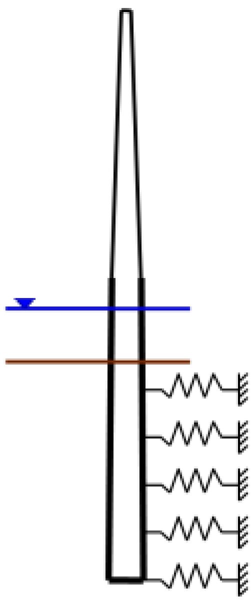

Figure 4: Distributed springs [15]

To determinate the stiffness of each spring, which is going to represent the different characteristics of each soil type, it has been considered the theory developed by Lysmer and Richart (1966) [16] [8] wich reads:

$$G_s = \frac{E_s}{2 \text{ x } (1+ \upsilon)} \quad (6)$$

$$k = \frac{16 \text{ x } D_0 \text{ x } (1- \upsilon) \text{ x } G_s}{7-8\upsilon} \quad (7)$$

Where, for the soil, G_s (N/m^2) is the shear modulus, E_s (N/m^2) is the young's modulus, υ is the poisson's coefficient, K (N/m) is the spring stiffness and D_0 is the monopile's outer diameter.

1.4.1.4 Range of natural frequencies

One of the most important consideration, to be taken into account when designing the structure of a wind turbine is the natural frequency [18] of the structure because they determine the dynamic behaviour of the offshore wind turbine.

Free vibration occurs when a mechanical system is set off with an initial input and then allowed to vibrate freely, the mechanical system will then vibrate at one or more of its "natural frequency" and damp down to zero.

The fundamentals of vibration analysis can be understood by studying the simple mass – spring – damper model. Indeed, even a complex structure such as an automobile body can be modeled as a "summation" of simple mass–spring–damper models.

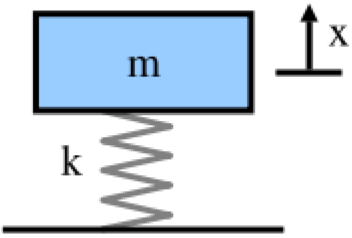

Figure 5: mass-spring-damper model [23]

To start the investigation of the mass–spring–damper we will assume the damping is negligible and that there is no external force applied to the mass [23].

The force applied to the mass by the spring is proportional to the amount the spring is stretched "x" (we will assume the spring is already compressed due to the weight of the mass). The proportionality constant, k, is the stiffness of the spring and has units of force/distance (eg lbf/in or N/m). The negative sign indicates that the force is always opposing the motion of the mass attached to it [23].

$$F_s = -kx. \tag{8}$$

The force generated by the mass is proportional to the acceleration of the mass as given by Newton's second law of motion [23].

$$\Sigma\, F = ma = m\ddot{x} = m\frac{d^2x}{dt^2}. \tag{9}$$

The sum of the forces on the mass then generates this ordinary differential equation [23]:

$$m\ddot{x} + kx = 0. \tag{10}$$

If we assume that we start the system to vibrate by stretching the spring by the distance of *A* and letting go, the solution to the above equation that describes the motion of mass is [23]:

$$x(t) = A\cos(2\pi f_n t). \qquad (11)$$

This solution says that it will oscillate with simple harmonic motion that has an amplitude of *A* and a frequency of f_n. The number f_n is one of the most important quantities in vibration analysis and is called the undamped natural frequency . For the simple mass–spring system, f_n. is defined as [23]:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m}}. \qquad (12)$$

Note: Angular frequency ω ($\omega = 2\pi f$) with the units of radians per second is often used in equations because it simplifies the equations, but is normally converted to “standard” frequency (units of Hz or equivalently cycles per second) when stating the frequency of a system.

If you know the mass and stiffness of the system you can determine the frequency at which the system will vibrate once it is set in motion by an initial disturbance using the above stated formula. Every vibrating system has one or more natural frequencies that it will vibrate at once it is disturbed [23].

If the frequency of excitation is near a natural frequency, resonance occurs and the resulting response will be larger than in the quasi-static case. This leads to higher stresses in the support structure and, more importantly to higher stress ranges, an unfavourable situation with respect to the fatigue life of the offshore wind turbine. Therefore it is important to ensure that the excitation frequencies with high energy levels do not coincide with a natural frequency of the support structure [23].

In the case of an offshore wind turbine, excitation is due to both wind and waves. According to Wybren de Vries [18], for fatigue considerations sea states with a high frequency of occurrence have the largest effect. These are generally relatively short waves with a significant wave height Hs of around 1 m to 1.5 m and a zero-crossing period Tz of around 4 s to 5 s.

The wind excitation frequencies that should be avoided are those that coincide with the range of rotational frequencies of the rotor. With a minimum rotational speed at the cut-in wind speed of 6.9 rpm and a maximum rotational speed of 12.1 rpm, the rotational frequency interval to stay clear of ranges from 0.222 Hz to 0.311 Hz. This interval is indicated with 1P.

Furthermore, the blade-passing frequency interval should also be avoided. This interval, indicated with 3P for a triple bladed turbine is equal to the rotational frequency interval times the number of blades. Taking the above into account, the first natural frequency is chosen at 0.29 Hz. The second natural frequency must be well above the 3P frequency range. Applying a 10% margin on the upper boundary of the 3P range the minimum second natural frequency is 0.666 Hz.

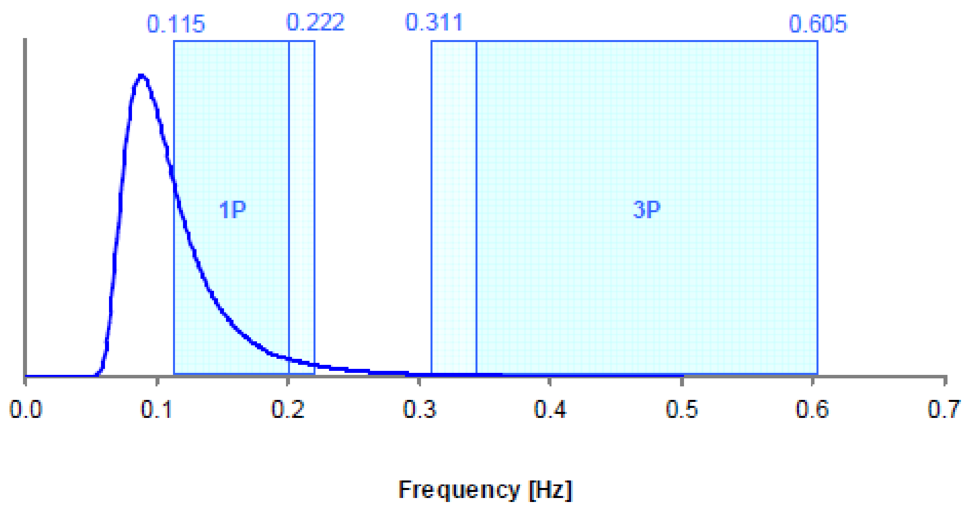

Figure 6: Diagram showing allowable frequency range and excitation frequencies [18]

1.5 THEORY IS BASED ON UPNA THESIS

We will compare the results we obtain with those obtained in [2] so because of that we will describe the theory in which this thesis is based on.

1.5.1 Theoretical basis of the boundary conditions

1.5.1.1 Wind

For the last load test, this project considers that the worst charge that can exist is a force of 800000 N, according to data provided by the company Acciona Winpower [2].

1.5.1.2 Water

The Morison equation [2] is used to calculate the hydrodynamic forces on slender bodies, such as members of the lattice. This means that the diameter of the cross section of the member, D, must be less than 1/5 of the wavelength of the incident wave, L.

$$\frac{1}{5} \geq \frac{D}{L}$$

For higher diffraction parameters apply other theories as:

- Froude-Krylov (if the inertial force predominates but the cylinder is still "small" relative to L).
- The theory of diffraction (if the size of the cylinder starts to be comparable to the wavelength).

The hydrodynamic force calculated by Morison`s equation is divided into a component for calculating the viscous drag and one that calculates the inertia loads on the structure of bars. Morison's theory comes from the Bernoulli equation for a more detailed view of this development go to the / reference 7 /, Annex D of IEC 61400-3. The equation for a static member is:

$$F = \frac{1}{2} C_d \rho D |U| U + C_m A \dot{U} \tag{13}$$

While:

- F is the force per unit length on the member;
- Cd is the drag coefficient, drag;
- Cm is the ratio of inertia;
- ρ is the density of water;
- D is the diameter of the member;
- A is the cross sectional area of the member;
- U is the velocity of flow normal to the member;
- U is the acceleration of the normal flow to the member;

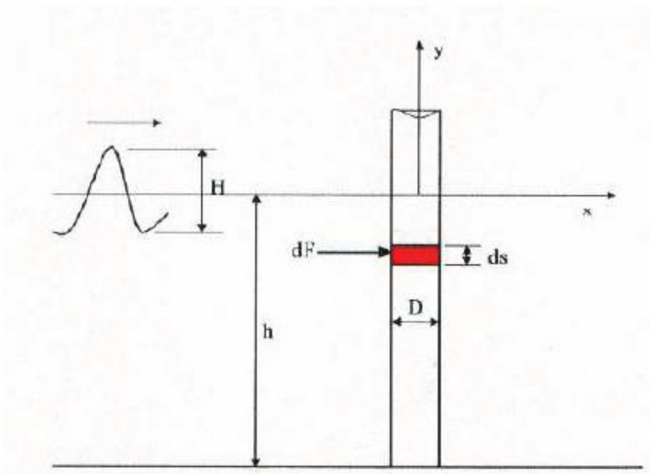

Figure 7: Definition of the wave load on the cylinder [2]

1.5.1.3 Soil

For modeling soil this thesis considers fixed interaction soil-structure

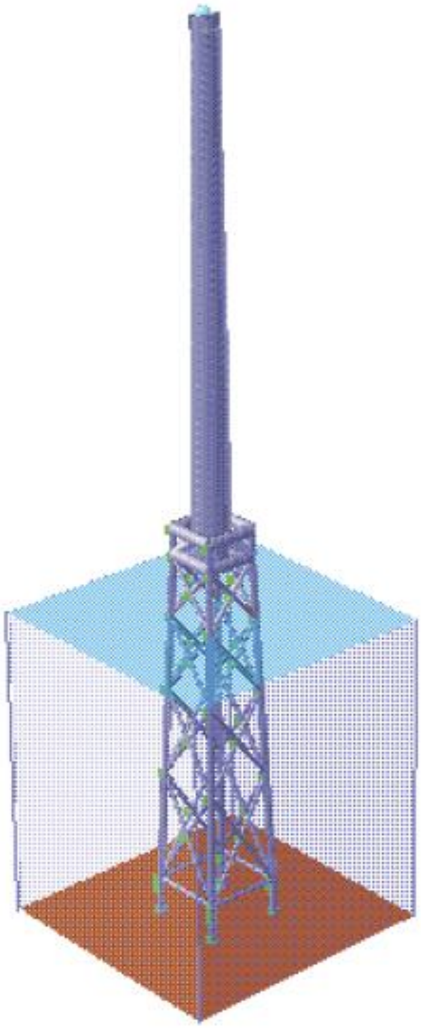

Figure 8: Jacket's structure [2]

1.5.1.4 Structure and dimensions

The whole structure [2] consists of a lattice of steel S355JR according to EN10025 UNE with the following dimensions and properties of the elements:

No	outer diameter	thickness
1	0.8	0.02
2	1.2	0.05
3	1.2	0.05
4	1.2	0.035
5	1.2	0.035
6	1.2	0.035
7	1.2	0.035
8	0.8	0.02
9	2	0.08
10	2.082	0.06
11	0.8	0.02
12	0.8	0.02
13	1.2	0.04
14	1.2	0.05
15	0.8	0.02
16	5.5885	0.032
17	5.4475	0.032
18	5.2	0.03
19	4.941	0.028
20	4.6825	0.024
21	4.447	0.022
22	4.2235	0.02
23	4.059	0.03

Table 2: Jacket's elements dimensions [2]

- Yield strength: σy = 356 MPa
- Density: ρ = 7850 kg / m3
- Young's modulus: E = 2.1 · 105 MPa
- Poisson ratio: ν = 0.3
- Thermal coefficient: $\alpha = 1.2 \cdot 10^{-5}\ °\ C^{-1}$

The total volume of the lattice base is 158 m^3, and the tower mass is 347460.35713 kg so the tower volume is 44.26 m^3. According to this the total volume of the structure is 202.26 m^3.

It considered as well a NREL 5MW wind turbine for the analysis

1.5.1.5 Range of natural frequencies

As already mentioned the valid frequency range is very important in shaping the design of wind turbines. According to it, the valid range of values which is defined based on the following chart [2]:

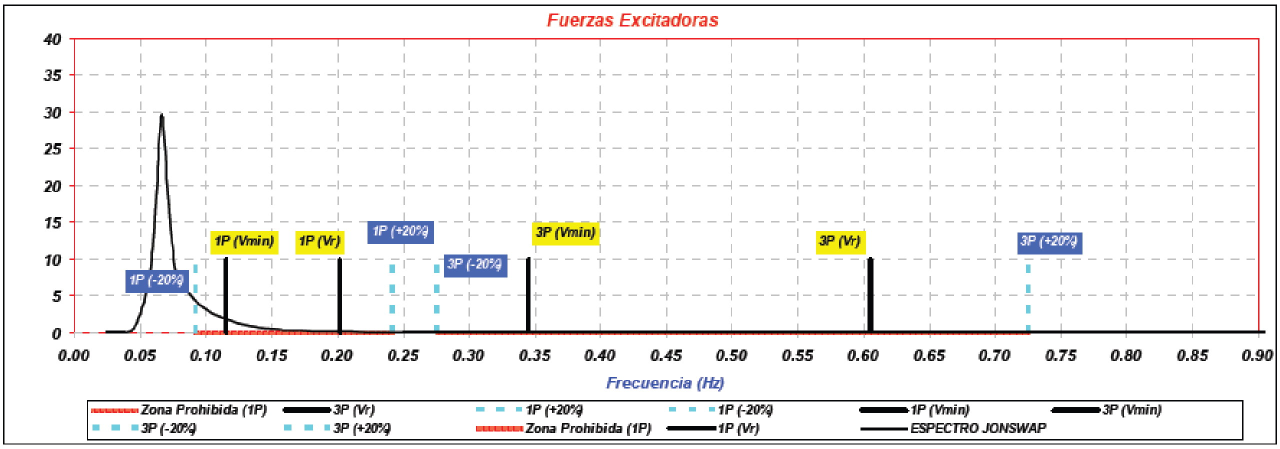

Figure 9: Frequencies of excitation forces in a 5mw wind turbine offshore [2]

Forbidden ranges are indicated in red are all frequencies at which excites the machine with the passage of a blade, 1P, and the three blades, 3P, from the minimum speed of rotation of the rotor, Vmin, until the cutting speed, Vr, maximum speed of rotation of the rotor. Also take into account a percentage of security marked by the certification, which should not be exceeded, ± 20%. Another forbidden range, and very important in the design of the marine substructure is the excitation of the sea, which is represented in this graph by the JONSWAP spectrum for T = 8s and H=15m.

2. MONOPILE-TYPE STRUCTURE

In this chapter we describe the properties of the wind turbine we have designed and therefore also the loads that the structure will be exhibited to the theories explained in Chapter 1 and because of its properties.

2.1. PROPERTIES AND SOIL INTERACTION

As mentioned in the chapter 1, the main objective of this project will be to study, design and optimization of a wind turbine monopile type, based on the analysis of the last load test. First we are going to determine the properties of the monopole strucuture. To do this, we will enter in engineering data and select a material. The material we will select will be structural steel and these will be his properties:

Properties of Outline Row 3: Structural Steel

	A	B	C
1	Property	Value	Unit
2	Density	7850	kg m^-3
3	Coefficient of Thermal Expansion		
4	Coefficient of Thermal Expansion	1,2E-05	C^-1
5	Reference Temperature	22	C
6	Isotropic Elasticity		
7	Young's Modulus	2E+11	Pa
8	Poisson's Ratio	0,3	

Table 3: Properties of the structural steel [2]

The first action undertaken to design the wind turbine that is dealt with, was a preliminary design given the indications of the size specified in Chapter 1 [7]. Once the preliminary design was completed, I performed a modal analysis to see if it is conform to the natural frequencies related specifications we have discussed [18].

Subsequently I was applying changes and repeating the modal analysis to get a wind turbine that satisfied all specifications required and which ultimately resulted as follows in a structure which total volume is 202.82 m^3:

Tower	Dimensions
Length	77 m
Top outer diameter	4 m
Bottom outer diameter	5,8 m
Thickness	0,1 m

Table 4: Tower dimensions

Transition piece	Dimensions
Length	12 m
Outer diameter	5,8 m
Thickness	0,1 m

Table 5: Ttransition piece dimensions

Monopile	Dimensions
Water depth	20 m
Length	38,5 m
Thickness buried part	0,05 m
Thickness underwater part	0,135 m
Outer diameter underwater part	5,81 m
Thickness of the transition piece introduced into	0,03 m
Length of the transition piece introduced into	4 m
Outer diameter (soft clay)	5,815 m
Length (soft clay)	6 m

Outer diameter (silty sand)	5,82 m
Length (silty sand)	2 m
Outer diameter (dense sand)	5,825 m
Length (dense sand)	2,5 m
Outer diameter (sand and gravel)	5,83m
Length (sand and gravel)	2,5 m
Outer diameter (weathered rock)	5,835 m
Length (weathered rock)	4,5 m
Outer diameter (rock)	5,84 m
Length (rock)	3 m

Table 6: Monopile dimensions

We also comment that in order to perform the modal analysis, the soil-monopile interaction needs to be defined, according to [15] [16]. This is achieved by elastic support, which is defined in the Ansys finite element program by selecting the area of the monopile which is in contact with the soil and wich is going to suffer such interaction between both. This elastic support requires the soil's stiffness values in N/m^3, something we can achieve from each spring stiffness, that make up the distributed springs model, using the following equation defined in the ANSYS program manual [20]:

$$\mathrm{K} = \frac{n\,x\,K}{A} \qquad (14)$$

Where n is the number of springs, A is the application area and k is each spring stiffness, that as it was defined in Chapter 1 [8] [16] is calculated as follows:

$$k = \frac{16 \text{ x } \mathrm{D_0} \text{ x } (1-\upsilon) \text{ x } \mathrm{G_s}}{7-8\upsilon} \qquad (7)$$

Where, for the soil, G_s (N/m^2) is the shear modulus, E_s (N/m^2) is the young's modulus, υ is the poisson's coefficient, K (N/m) is the spring stiffness and D_0 is the monopile's outer diameter.

Considering in (14) a spring per area unit, we will obtain the soil stiffness per unit of measure.

Finally, according to the distributed spring model [15] [16] explained in Chapter 1, where we consider the interaction soil-monopile by a distributed spring model, which will represent in terms of their stiffness the different soil layers considered [1], we will obtain different stiffness values for each stratum defined in our considered soil model, that can be seen in the table 1 below.

Depth (m)		**Material**
From	**To**	
0,0	5,0-6,7	Very soft to soft clay
5,0-6,7	7,0-9,5	Loose silty sand to silty sand
7,0-9,5	10,0-10,7	Medium dense sand and stiff sandy clay
10,0-10,7	11,1-15,0	Sand and gravel alternating with gravel and cobbles
11.1-15,0	17,0	Moderately weathered rock to completely weathered rock
17,0	28	Rock

Table 1: Seabed stratification [1]

We use intermediate values of depth of each layer according to the table 1 and intermediate values of the parameters that define the characteristics of each soil layer considered as the values defined in [3], for not having exact values of a particular place.

Accordingly we obtain the corresponding values of the springs of each layer of soil and subsequently from them, the value of the foundation stiffness of each layer of soil as shown below.

- **Soft clay:**
  - Es = 1- 3 MPa → Es = 2 MPa
  - $\upsilon = 0.3$
  - $D_0 = 5.815$

$Gs = 769230.7692$ Pa

$K = 10890969.9$ N/m

K = 10890969.9 N/m^3

- **Silty sand:**
  - Es = 10 - 28 MPa → Es = 20 MPa
  - $\upsilon = 0.3$
  - $D_0 = 5.82$

$Gs = 7692307.692$ Pa

$K = 109003344.5$ N/m

K = 109003344.5 N/m^3

- **Dense sand:**
  - Es = 35 - 69 MPa → Es = 52,5 MPa
  - $\upsilon = 0.3$
  - $D_0 = 5.825$

$Gs = 20192307.69$ Pa

$K = 286379598$ N/m

K = 286379598 N/m^3

- **Sand and gravel:**
  - Es = 70 - 170 MPa → Es = 120 MPa
  - υ = 0.3
  - D_0 = 5.83

Gs = 48000000 Pa

K = 681349565.2 N/m

K = 681349565.2 N/m^3

- **Weathered rock:**
  - Es = 1- 20 GPa → Es = 10 GPa
  - υ = 0.3
  - D_0 = 5.835

Gs = 3846153846 Pa

K = 5.464214047 x 10^{10} N/m

K = 5.464214047 x 10^{10} N/m^3

- **Rock:**
  - Es = 10 - 70 GPa → Es = 40 GPa
  - υ = 0,3
  - D_0 = 5,84

Gs = 1.538461538 x 10^{10} Pa

K = 2.187558528 x 10^{11} N/m

K = 2.187558528 x 10^{11} N/m^3

In summary the soil model we have obtained according to the considerations, a discussed earlier, can be seen in the following table:

Seabed stratification	**Depth (m)**	**Foundation stiffness (N/m^3)**
Soft clay	6	10890969.9
Silty sand	2	109003344.5
Dense sand	2.5	286379598
Sand and gravel	2.5	681349565.2
Weathered rock	4.5	$5.464214047 \times 10^{10}$
Rock	3	$2.187558528 \times 10^{11}$

Table 7: Seabed stratification model

2.2. FORCES SUPPORTED BY THE STRUCTURE

To calculate the loads we will use the theory described in Chapter 1 in section 4 and also the value of the parameters considered in the thesis used to compare, which are shown in the next section:

2.2.1. Parameters

2.2.1.1. 5 MW NREL wind turbine

As discussed in Chapter 1, we will use NREL 5MW wind turbine to make it possible to compare the results with the selected thesis, so that the values that we consider are [18]:

Vmax	25 m/s
Ω	12.1 rpm
Rotor radius	63 m
Nacelle's mass	240000 kg
Blades mass	110000 kg

Table 8: 5 MW NREL wind turbine characteristics

2.2.1.2. Water

As with the wind turbine, we will consider defined values as the worst in the thesis of UPNA [2]:

ρ_{wa}	1025 kg/m^3
Hs	13.7 m
T	12.3 s

Table 9: Water conditions

2.2.1.3. Wind

ρ_{wind}	1.226 kg/m^3

Table 10: Wind conditions

From the data in [2] is can be seen that the worst case scenario is used on the wind turbine due to wind is 800000 N according to data provided by Acciona windpower [2], but we will analyze as this force and the other taking into account the characteristics of air and the theories of Chapter 1.

2.2.2. Supported loads

As discussed in previous sections, we will calculate the most unfavorable conditions that the wind turbine may be exhibited to, for perform the last load test, that is the analysis of the response that will have the offshore wind turbine, under the worst possible environmental conditions. This involves calculating the maximum force that can have the wind on the structure and also the maximum force that can exert the sea

2.2.2.1. Wind force

Considering the values defined in the previous section and the theory of Chapter 1 [6], the worst force by wind is:

- $F_T = 0.5 \text{ x } \rho_{wa} \text{x } \pi R^2 \text{ x } V_1 \text{ x } C_T(\lambda)$
  - $\lambda = \frac{\Omega \, x \, R}{V_1}$ = 3.1931148
  - $C_T(\lambda)$ = 0.3
  - F_T = 1433153.532 N

2.2.2.2. Water force

Taking into account the values that have been considered as the most unfavorable case of the sea and the theory of Chapter 1 [6], the worst forces by the sea are:

- $F_D = \frac{C_D \text{ x } \rho_{wa} \text{ x } (2R) \text{x } H_s{}^2 \text{ x } \omega^2}{16\psi} \text{ x } \frac{\sinh(2\psi d_w) + 2\psi d_w}{\cosh(2\psi d_w) - 1}$
  - $C_D = 0.7$
  - $d_w = 20$
  - $\omega = \frac{2\pi}{T} = 0{,}51083$
  - $\psi = \frac{\omega^2}{g} = 0.02663$
  - Hs = 13.7
  - T = 12.3

F_D = 1799356.458 N

- $F_M = \frac{\pi \text{ x } C_M \text{ x } \rho_{wa} \text{ x } (2R)^2 \text{x Hs x } \omega^2}{8\psi}$
  - $C_M = 2$

F_M = 3635565.083 N

Finally the worst force exerted by the sea in the most unfavorable conditions will be:

- $F_H = \max\ \{F_D \cos^2(-\omega t) + F_M \sin(-\omega t)\} \quad \text{for } -\frac{T}{4} \leq t \leq 0$

Considering the value of T as defined in this chapter in section 2, we note the possible values of t which obtain the maximum value of F_H as follows:

T = 12.3 s → - 3.075 ≤ t ≤ 0

- t = 0 → $\cos^2(-\omega t) = 1$

- t = -3.075 → $\sin(-\omega t) = 1$

So the highest value of F_H will be:

F_H = 3635565.083 N

To sum, the worst possible environmental conditions that we consider in the last load test can be seen in figure 10, shown below.

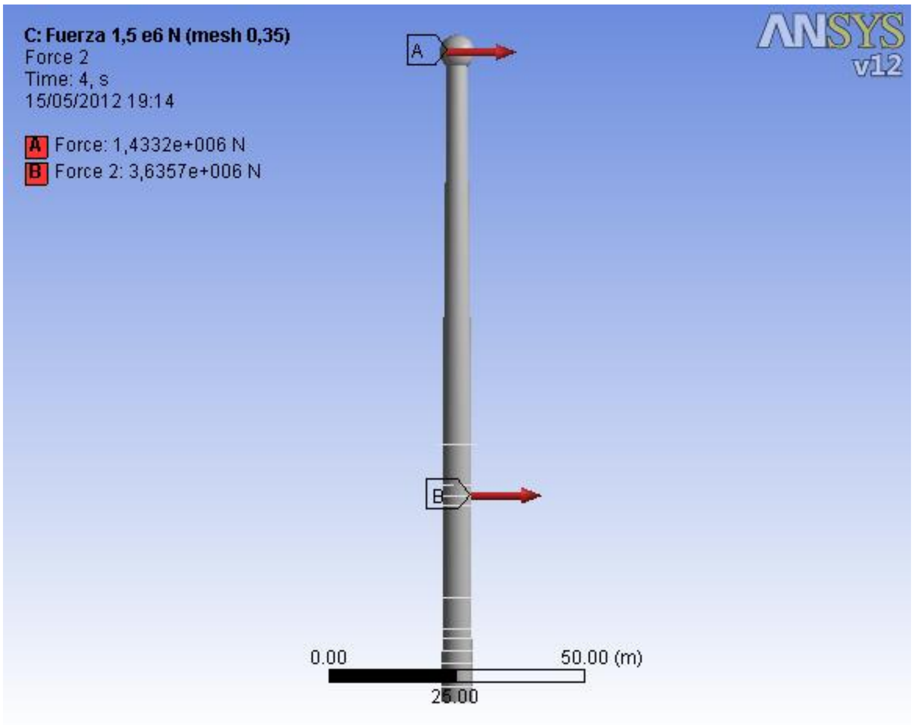

Figure 10: Forces discussed in the last load test

3. ANALYSIS

3.1. MODAL ANALYSIS

As discussed in the previous chapter, the first analysis is a modal analysis. This analysis is particularly important when designing the offshore monopile structure as it gives us the natural frequencies of the structure, that are going to define the dynamic behavior of it.

To perform this analysis we define first the soil-monopile interaction as discussed above.

Subsequently we perform a meshing of the structure defined by an element size of 0.4. This is considered well after trying several element sizes and show that the variation in results with smaller sizes were very small, it is also necessary to mention that for smaller sizes the computer needs a long calculation time for analysis and often memory error was reached in Ansys.

The difference of values obtained for different sizes of element shown in the following table:

Mode	**Frequency [Hz] (element size 0,35)**	**Frequency [Hz] (element size 0,4)**	**Error**
1	0.28086	0.28093	0.025 %
2	0.28087	0.28095	0.028 %
3	1.6915	1.6916	0.0059 %
4	1.6915	1.6916	0.0059 %

Table 11: Comparison of values based on element size

We found that the values of both analyzes are very similar and very small errors are obtained, also, as discussed above for analysis with smaller element sizes the computer need long time for calculation and sometimes crashed so we consider this element size as quite adequate.

Can be seen in the following table the properties of the suitable mesh.

Statistics	
Nodes	161871
Elements	80805
Mesh Metric	None

Table 12: Mesh statics properties of the structure

3.1.1 Natural frequencies of monopile structure

For comparison and analysis of results we have considered the first 4 natural frequencies. We have considered these first 4 frequencies because they are the most important in the design of the structure as discussed in Chapter 1, showing in figure 6.

The natural frequencies in the modal analysis are:

Mode	Frequency [Hz]
1,	0.28093
2,	0.28095
3,	1.6916
4,	1.6916

Table 13: Natural frequencies of monopile structure

Below we see the corresponding figures for each mode:

Mode 1:

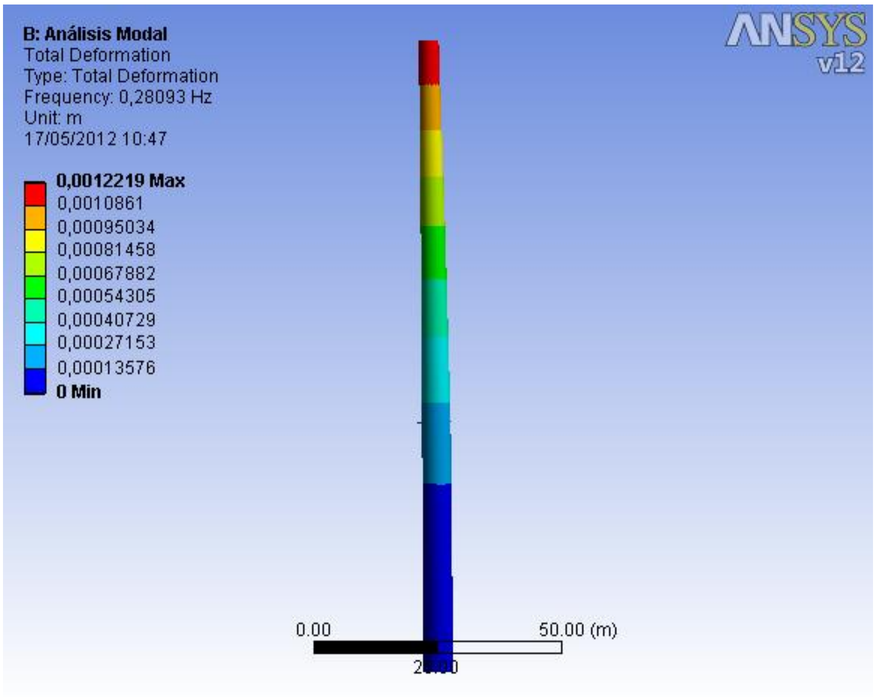

Figure 11: Mode 1 of vibration of monopile structure at 0.28093 Hz

Mode 2:

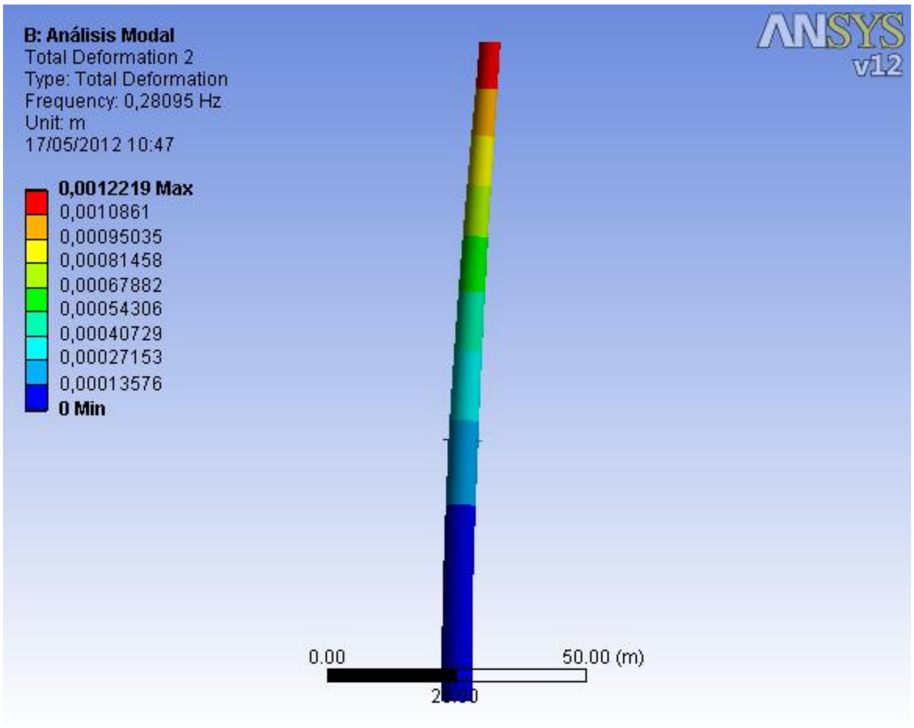

Figure 12: Mode 2 of vibration of monopole structure at 0.28095 Hz

Mode 3:

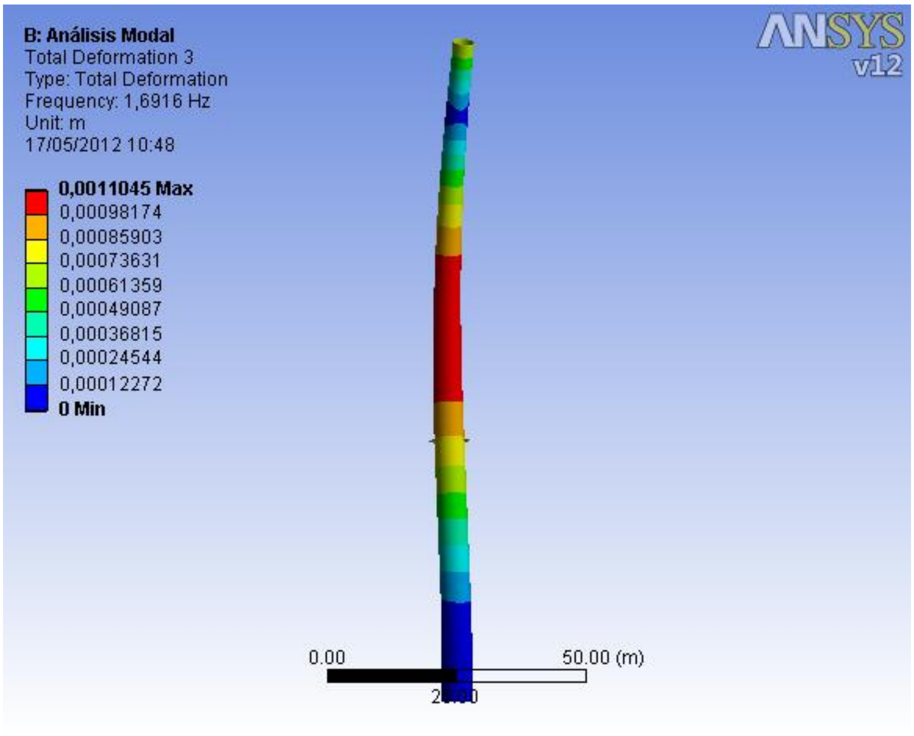

Figure 13: Mode 3 of vibration of monopile structure at 1.6916 Hz

Mode 4:

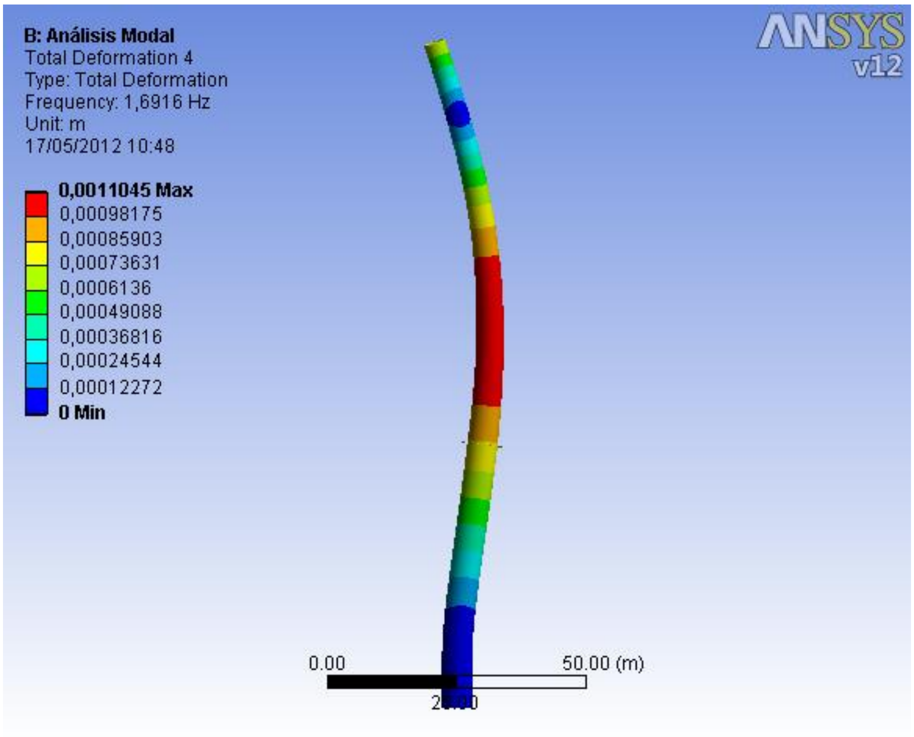

Figure 14: Mode 4 of vibration of monopile structure at 1.6916 Hz

Observing the chart below:

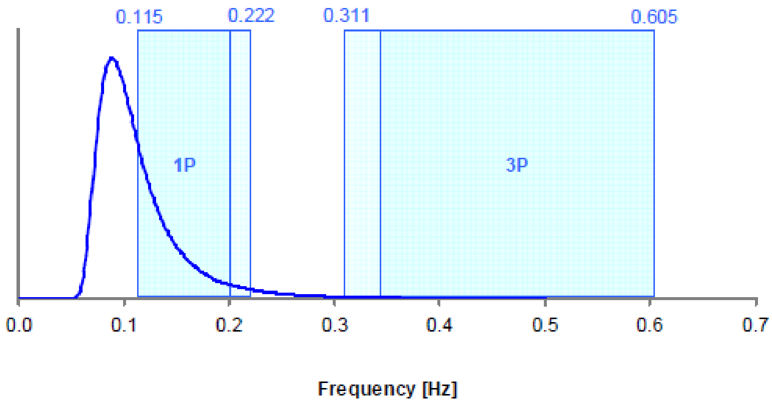

Figure 6: Diagram showing allowable frequency range and excitation frequencies [18]

When observing Figure 6, one can verify that the first two frequencies are within the limits set by the specifications which are 0.222 Hz - 0.3111 Hz and both following are far from the minimum limit which is 0.605 Hz, so that we can now consider the design made as quite acceptable and appropriate.

3.1.2 Natural frequencies of jacket structure

We will now analyze the results of the thesis for the design of wind turbine support structure consisting of a jacket type.

In the table below we can see the results for this thesis in modal analysis [2]:

Mode	Frequency [Hz]
1,	0.29847
2,	0.30884
3,	1.15087
4,	1.18043

Table 14: Natural frequencies of the jacket type structure

Below we can see the corresponding figures for each mode:

Mode 1:

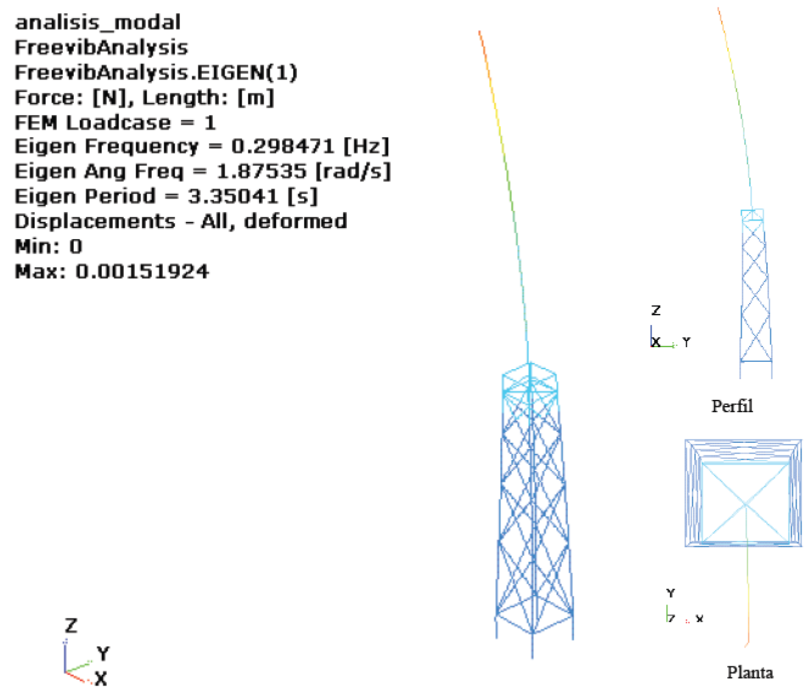

Figure 15: Mode 1 of vibration of jacket structure at 0.29847 Hz [2]

Mode 2:

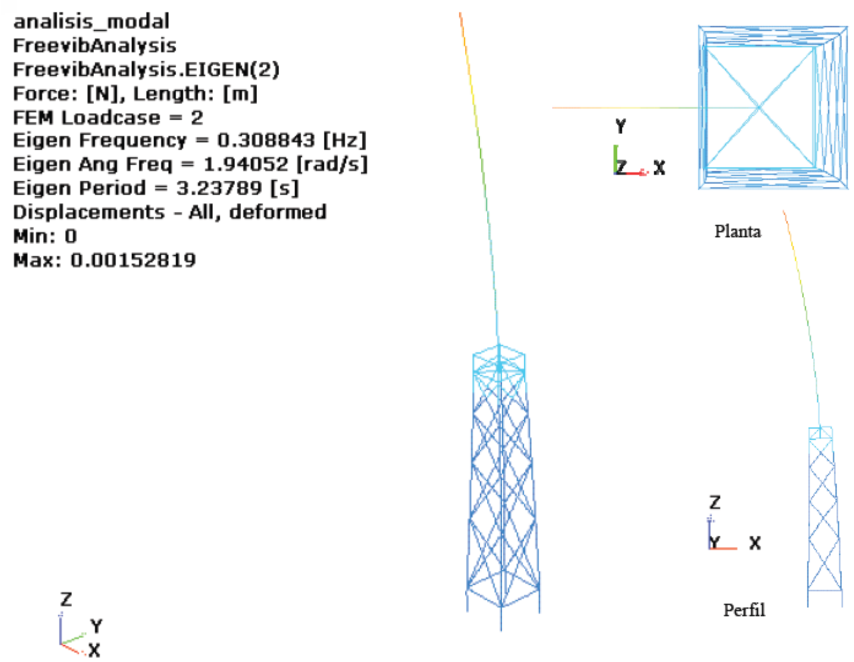

Figure 16: Mode 2 of vibration of jacket structure at 0.308843 Hz [2]

Mode 3:

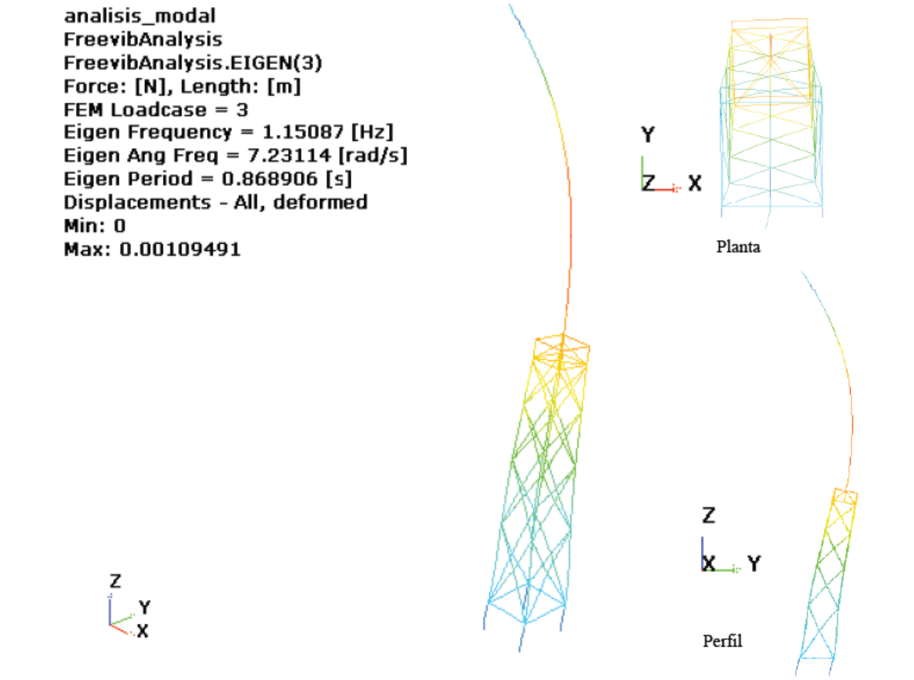

Figure 17: Mode 3 of vibration of jacket structure at 1.15087 Hz [2]

Mode 4:

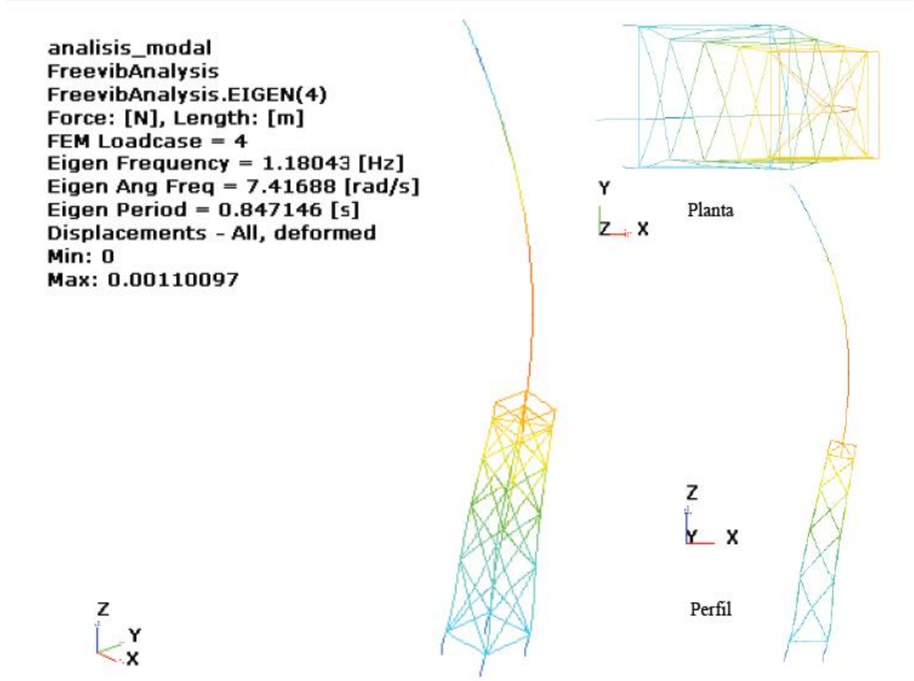

Figure 18: Mode 4 of vibration of jacket structure at 1.18043 Hz [2]

Comparing the results of my analysis and the results of the jacket-type wind turbine design [2], which are visualized in table 15, we see that the first two natural frequencies of both are quite similar, however, the next two frequencies shows a significant difference. This difference is because the values obtained in my thesis are further away from the limit imposed by the specifications which is 0.605, so that we could consider as more reliable, because it has a greater safety margin.

Monopile structure		**Jacket-type structure [2]**	
Mode	**Frequencies (Hz)**	**Mode**	**Frequencies (Hz)**
1	0.28093	1	0.29847
2	0.28095	2	0.30884
3	1.6916	3	1.15087
4	1.6916	4	1.18043

Table 15: Frequencies comparison between monopole structure and jacket-type structure [2]

3.2 LAST LOAD TEST

As discussed in chapter 2 in section 2.2, the last load test is the analysis of the response that will have the offshore wind turbine, under the worst possible environmental conditions. This involves calculating the maximum force that can have the wind on the structure and also the maximum force that can exert the sea.

The last load test is been made in two different ways, first considering the worst possible conditions based on the theory of Chapter 1 [6], that can be seen as follows:

Vmax	25 m/s
Ω	12.1 rpm
Rotor radius	63 m
Nacelle's mass	240000 kg
Blades mass	110000 kg

Table 8: 5 MW NREL wind turbine characteristics [18]

ρ_{wa}	1025 kg/m^3
Hs	13.7 m
T	12.3 s

Table 9: Water conditions [2]

ρ_{wind}	1.226 kg/m^3

Table 10: Wind conditions [2]

and then as the worst wind force considered in the thesis jacket type structure [2], in which is analyzed with a maximum wind force of 800000 N.

Within the last load test we will analyze the deformation suffered by the structure.

3.2.1 Monopile structure total deformation

3.2.1.1 According to calculated maximum load

In this last load test the applied forces are the ones calculated in chapter 2, based on the theory described in Chapter 1 and on the worst possible environmental conditions which are defined above.

According to the values of these forces are:

F_T = 1433153.532 N (wind)

F_H = 3635565.083 N (water)

The applications of these loads in the monopole structure are shown in figure 10.

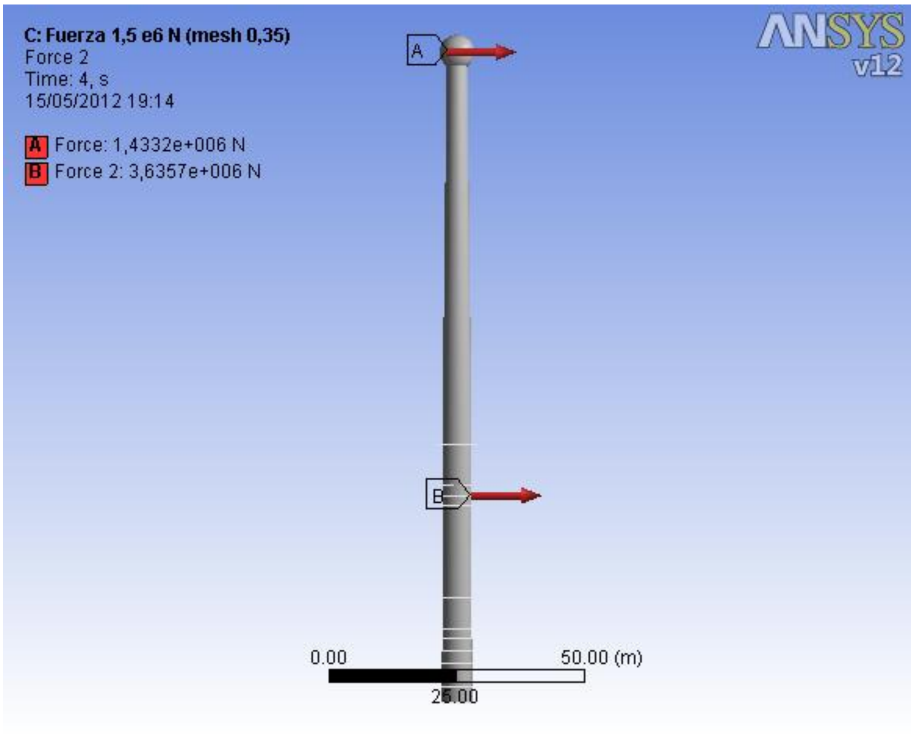

Figure 10: Forces discussed in the last load test

The deformation suffered by the structure due to these loads can be seen in figure 19:

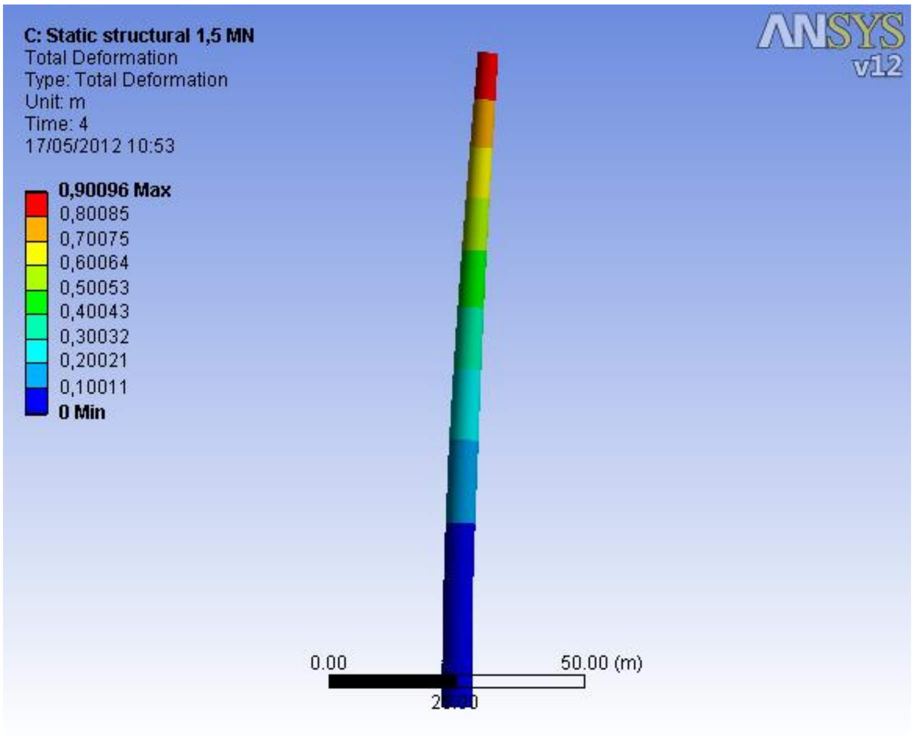

Figure 19: Total deformation of monopile structure because of last load test by maximum calculated load

The maximum deformation occurs at the highest point of the structure as expected and the value is 0.90 m. It must be considered it is very difficult for both conditions to occur simultaneously besides being very difficult to produce either separately situations.

Accordingly it is appropriate to say that this is a very sturdy design because the deformation suffered represents not even 1% of its total length.

3.2.1.2 According to 800000 N wind force

To perform this last load test we consider as most unfavorable wind load a load of 800000 N, since according to the thesis of the jacket type structure [2], the maximum load that can occur in offshore wind turbine with these characteristics is of 800000 N basing on data given by the company Acciona windpower [2].

In the figure 20 the applied loads considered in this analysis are visualized.

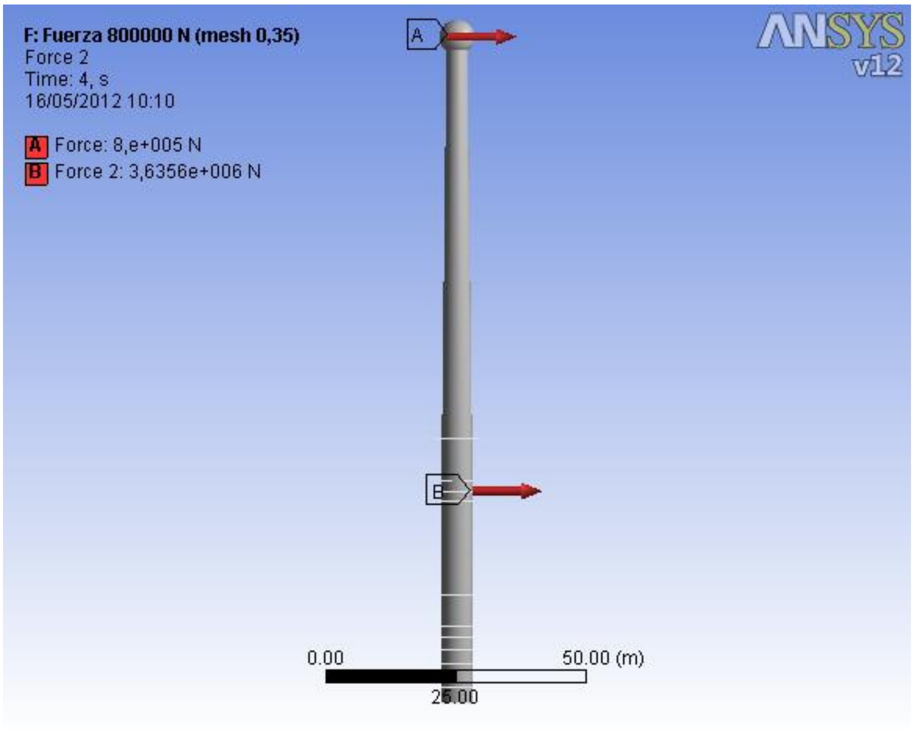

Figure 20: Forces applied in last load test basing on highest wind force of 800000 N

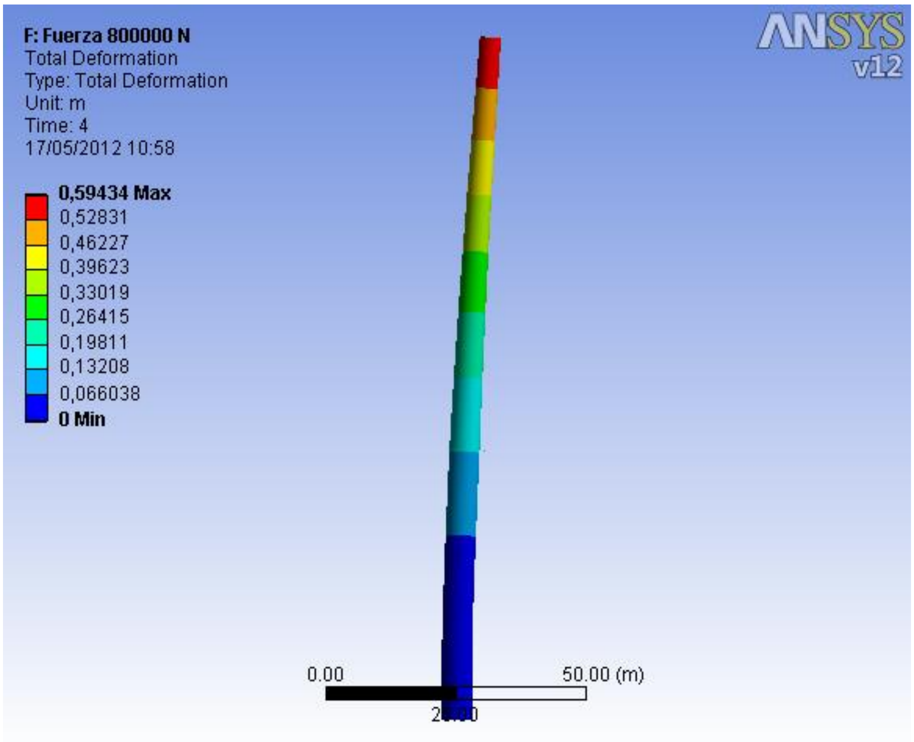

Figure 21: Total deformation of monopile structure because of last load test by 800000 N wind load

In the figure 21, the deformation suffered by the wind turbine under the influence of the load of 800000 N and the load corresponding to the sea is visualized. In this case the maximum deformation suffered by the structure has a value of 0.6 m.

3.2.2 Jacket structure total deformation

The last load test of the jacket structure [2] has been made taking consider a maximum wind load of 800000 N, based on the data provided by the company Acciona windpower, who claim that this is the maximum load corresponding to wind that a wind turbine may suffer with the characteristics described in previous sections.

According to these considerations a maximum deformation, at the highest point of the structure obtained a, value of 0.61 m, as we see in the figure 22:

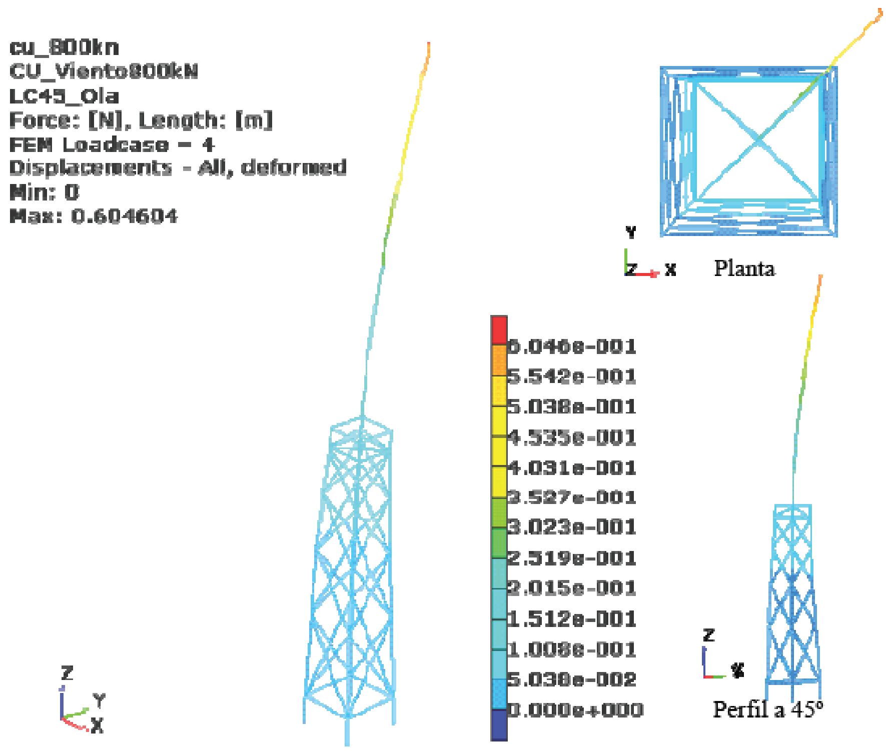

Figure 22: total deformation (m) testing of last load of the jacket structure [2]

Comparing the results of the two thesis, we can appreciate a slight decrease of the deformation, because in the structure of monopole type is obtained deformation value 0.60 m while in the jacket type is obtained 0.61 m. Considering the total volume of both structures, in the case of the monopile type 202.82 m^3 and jacket type 202.26 m^3

we can affirm that we have two structures that respond very similarly at the same excitation conditions.

Therefore we should also highlight that they are completely different characteristics structures. Since one is based on a single base structure monopile type, which makes it much easier the construction and subsequent installation, and the other is a lattice structure with a wide variety of tubes that difficult greatly the construction and installation.

Finally, one might conclude that in this aspect is more highly recommended and appropriate the monopile structure, because responses of the structure are obtained are the same or even better, using the same amount of material with a simpler design, which allows a better manufacturing and installation

4. OPTIMIZATION

4.1 OPTIMIZATION

Design optimization is a technique that aims to determine the best design or the optimal design. An "optimal design" is one that meets a number of specific requirements, with a minimum cost of certain factors, such as weight, area, volume, effort, etc. Virtually any aspect of your design can be optimized: dimensions (such as thickness), shape (such as fillet radii), placement of supports, cost of fabrication, natural frequency, material property, and so on. In other words, the optimal design is usually the one who manages to "be as effective as possible". In our case, we want to get the minimum total deformation as possible, using the fewest material [21] [22], but keeping the natural frequencies within the limits set by the specifications [18].

4.1.1. Optimization Methods

Optimization methods are traditional techniques that strive to minimize a function (objective function) that is subject to restrictions. In the ANSYS program are available two methods of optimization, the subproblem approximation method and first order method [20].

- The subproblem approximation method is an advanced zero-order method that use approximations (curve fitting) for all dependent variables (SV and the objective function). It is a general method that can be applied effectively to a wide range of engineering problems [20].
- The method of first order is that which uses the information of the derivative, i.e., the gradients of the dependent variables with respect to the design. The method is very accurate, is more suitable for problems that require high accuracy. However, this method can be computationally expensive [20].

For both the subproblem approximation and first order methods, the program performs a series of analysis-evaluation-modification cycles. That is, an analysis of the initial design is performed, the results are evaluated against specified design criteria, and the design is modified as necessary. The process is repeated until all specified criteria are met.

Understanding the algorithm used by the program is always useful, particularly for optimization. Below are details of the two optimization techniques in ANSYS [20].

4.1.1.1. The subproblem approximation method

The subproblem approximation method can be described as an advanced zero-order method in that it requires only the values of the dependent variables, and not their derivatives. There are two concepts that play a key role in the subproblem approximation method: the use of approximations for the objective function and state variables, and the conversion of the constrained optimization problem to an unconstrained problem [20].

4.1.1.2. First Order Method

Like the subproblem approximation method, the first order method converts the problem to an unconstrained one by adding penalty functions to the objective function. However, unlike the subproblem approximation method, the actual finite element representation is minimized and not an approximation [20].

The first order method uses gradients of the dependent variables with respect to the design variables. For each iteration, gradient calculations (which may employ a steepest descent or conjugate direction method) are performed in order to determine a search direction, and a line search strategy is adopted to minimize the unconstrained problem [20].

Thus, each iteration is composed of a number of subiterations that include search direction and gradient computations. That is why one optimization iteration for the first order method performs several analysis loops [20].

4.1.2. Goal driven optimization

GDO can be used for design optimization in three ways: the Screening approach, the MOGA approach, or the NLPQL approach. The Screening approach is a non-iterative direct sampling method by a quasi-random number generator based on the Hammersley algorithm. The MOGA approach is an iterative Multi-Objective Genetic Algorithm, which can optimize problems with continuous input parameters. NLPQL is a gradient based single objective optimizer which is based on quasi-Newton methods [20].

MOGA is better for calculating the global optima while NLPQL is a gradient-based algorithm ideally suited for local optimization. So you can start with Screening or MOGA to locate the multiple tentative optima and then refine with NLPQL to zoom in on the individual local maximum or minimum value. Problems with mixed parameter types (i.e., usability, discrete, or scenario parameters with continuous parameters) or discrete problems cannot currently be handled by the MOGA or NLPQL techniques, and in these cases you will only be able to use the Screening technique [20].

Usually the Screening approach is used for preliminary design, which may lead you to apply the MOGA or NLPQL approaches for more refined optimization results [20].

4.1.2.1. Screening (shifted Hammersley)

The shifted Hammersley method is the sampling strategy used for the Screening process. The conventional Hammersley sampling algorithm is a quasi-random number generator which has very low discrepancy and is used for quasi-Monte Carlo simulations. A low-discrepancy sequence is defined as a sequence of points that approximate the equidistribution in a multi-dimensional cube in an optimal way. In other words, the design space is populated almost uniformly by these sequences and, due to the inherent properties of Monte Carlo sampling, dimensionality is not a problem (i.e., the number of points does not increase exponentially with an increase in the number of input parameters). The conventional Hammersley algorithm is constructed by using the radical inverse function. Any integer n can be represented as a sequence of digits n_0, n_1, n_2, ..., n_m by the following equation [20]:

$$n = n_0 \, n_1 \, n_2 \, n_3 \cdots n_m \tag{15}$$

In general, for a radix R representation, the equation is [20]:

$$n = n_m + n_{m-1} * R + \cdots + n_0 \tag{16}$$

The inverse radical function is defined as the function which generates a fraction in (0, 1) by reversing the order of the digits in (3) about the decimal point, as shown below [20].

$$\begin{aligned}\Phi_R(n) &= 0n_m n_{m-1} n_{m-2} \cdots n_0 \\ &= n_m * R^{-1} + n_{m-1} * R^{-2} + \cdots + n_0 * R^{-(m-1)}\end{aligned} \qquad (17)$$

Thus, for a k-dimensional search space, the Hammersley points are given by the following expression [20]:

$$H_k(i) = [i/N, \Phi_{R1}(i), \Phi_{R2}(i), \cdots, \Phi_{Rk-1}(i)] \qquad (18)$$

Where i = 0... N indicates the sample points. Now, from the plot of these points, it is seen that the first row (corresponding to the first sample point) of the Hammersley matrix is zero and the last row is not 1. This implies that, for the k-dimensional hypercube, the Hammersley sampler generates a block of points that are skewed more toward the origin of the cube and away from the far edges and faces. To compensate for this bias, a point-shifting process is proposed that shifts all Hammersley points by the amount below [20]:

$$A = \frac{1}{2} N \qquad (19)$$

4.1.2.2. Multi-Objective Genetic Algorithm (MOGA)

The MOGA used in GDO is a hybrid variant of the popular NSGA-II (Non-dominated Sorted Genetic Algorithm-II) based on controlled elitism concepts. Currently, only continuous problems can be solved. The Pareto ranking scheme is done by a fast, non-dominated sorting method that is an order of magnitude faster than traditional Pareto ranking methods. The constraint handling uses the same non-dominance principle as the objectives, thus penalty functions and Lagrange multipliers are not needed. This also ensures that the feasible solutions are always ranked higher than the infeasible solutions [20].

The first Pareto front solutions are archived in a separate sample set internally and are distinct from the evolving sample set. This ensures minimal disruption of Pareto front patterns already available from earlier iterations. You can control the selection pressure (and, consequently, the elitism of the process) to avoid premature convergence by altering the parameter Percent Pareto [20].

4.1.2.3. Sequential Quadratic Programming (NLPQL)

NLPQL (Non-linear Programming by Quadratic Lagrangian) is a mathematical optimization algorithm as developed by Klaus Schittkowski. This method solves constrained nonlinear programming problems of the form [20].

Minimize:

Objective function $f = f(\{x\})$ (20)

Subject to:

Design variables (DV) $g_k(\{x\}) \leq 0, \forall k = 1,2,\ldots,K$ (21)

$h_l(\{x\}) \leq 0, \forall l = 1,2,\ldots,L$ (22)

Where:

State variables (SV) $\{x_L\} \leq \{x\} \leq \{x_U\}$ (23)

It is assumed that objective function and constraints are continuously differentiable. The idea is to generate a sequence of quadratic programming subproblems obtained by a quadratic approximation of the Lagrangian function and a linearization of the constraints. Second order information is updated by a quasi-Newton formula and the method is stabilized by an additional (Armijo) line search [20].

Newton's iterative method

Before the actual derivation of the NLPQL equations, Newton's iterative method for the solution of nonlinear equation sets is reviewed. Let f(x) be a multivariable function such that it can be expanded about the point x in a Taylor's series [20].

$$f(x + \Delta x) \approx f(x) + \{\Delta x\}^T \{f'(x)\} + \left(\frac{1}{2}\right) \{\Delta x\}^T [f''(x)] \{\Delta x\} \quad (24)$$

Where, it is assumed that the Taylor series actually models a local area of the function by a quadratic approximation. The objective is to devise an iterative scheme by linearizing the vector (Eq.24). To this end, it is assumed that at the end of the iterative cycle, the (24) would be exactly valid. This implies that the first variation of the following expression with respect to Δx must be zero [20].

$$\phi(\Delta x) = f(x + \Delta x) - \left(f(x) + \{\Delta x\}^T \{f'(x)\} + \left(\frac{1}{2}\right)\{\Delta x\}^T [f''(x)]\{\Delta x\} \right) \tag{25}$$

This implies that:

$$f(x + \Delta x)_{,\Delta x} - (\{f'(x)\} + [f''(x)]\{\Delta x\}) = 0 \tag{26}$$

The first expression indicates the first variation of the converged solution with respect to the increment in the independent variable vector. This gradient is necessarily zero since the converged solution clearly does not depend on the step-length. Thus, (26) can be written as the following [20]:

$$\{x_{j+1}\} = \{x_j\} - [f''(x_j)]^{-1}\{f'(x_j)\} \tag{27}$$

Where, the index "j" indicates the iteration (27) is thus used in the main quadratic programming [20].

4.2. OPTIMIZATION PROCESS

4.2.1. Initial situation

After obtain the results of the natural frequencies and total deformation we tried to optimize the design of the wind turbine structure. Starting with the initial design, we are going to modify the thickness of the structure to try to reduce the total deformation and the volume of the structure, but taking into account that two first natural frequencies will be within the range of specifications [18] defined in Chapter 1.

In the optimization process we have three different thicknesses to optimize. These three different thicknesses correspond with tower and transition piece thickness, with underwater part thickness and with underground part thickness.

At first, the values of these three different thicknesses, discussed in Chapter 3 in the first design of the monopole-type wind turbine, are:

Thickness	**Value (m)**
Tower and transition piece	0.1
Underwater part	0.135
Underground part	0.05

Table 16: Initial thicknesses of the structure

In the following figures these thicknesses are visualized.

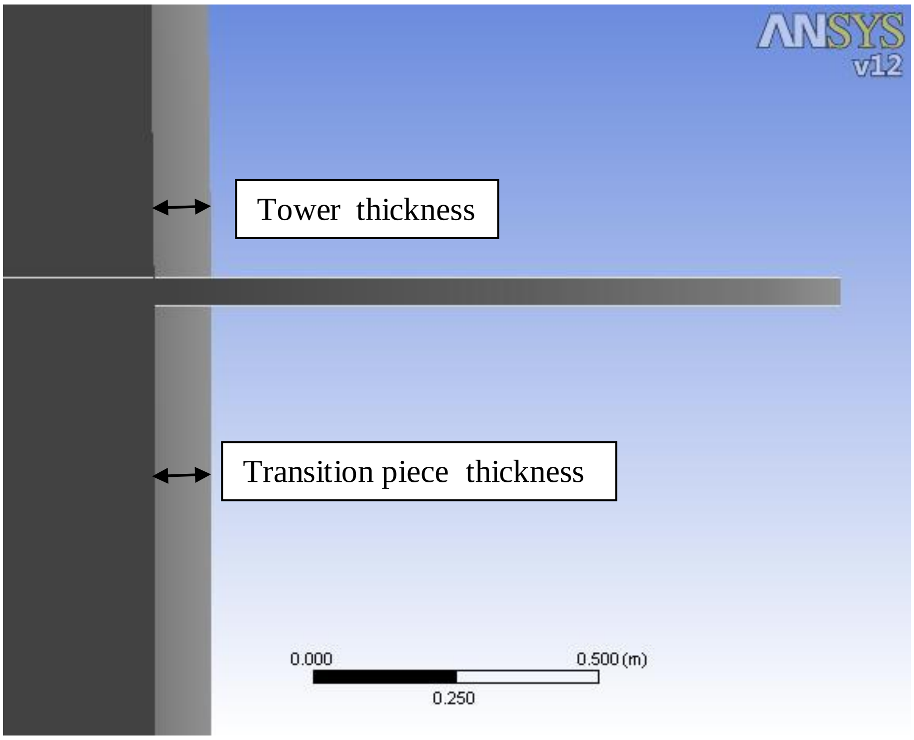

Figure 23: Tower and transition piece thicknesses

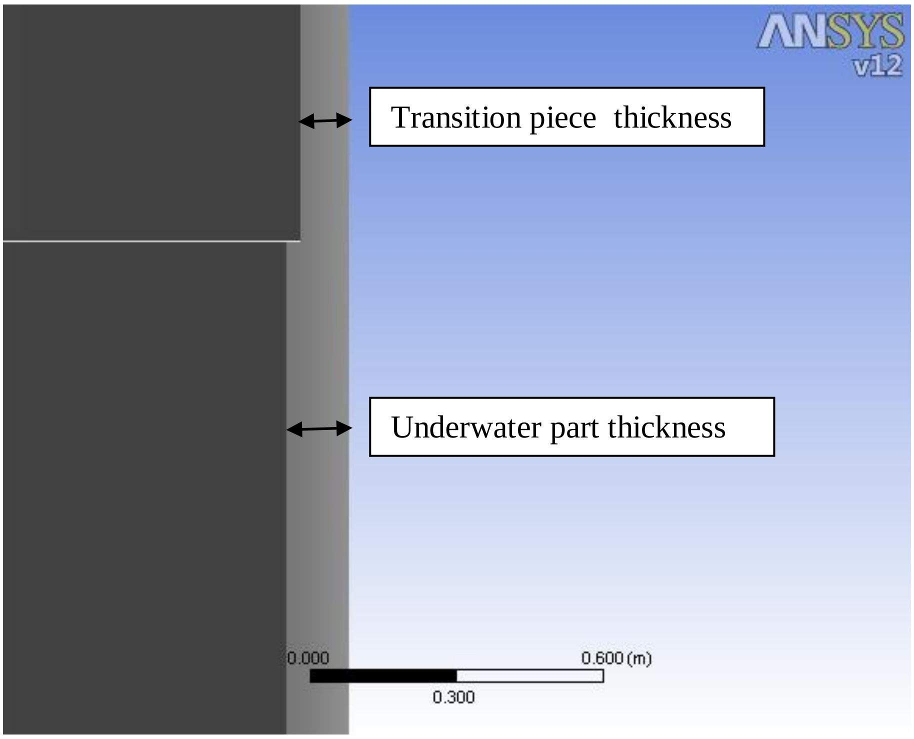

Figure 24: Transition piece and underwater part thicknesses

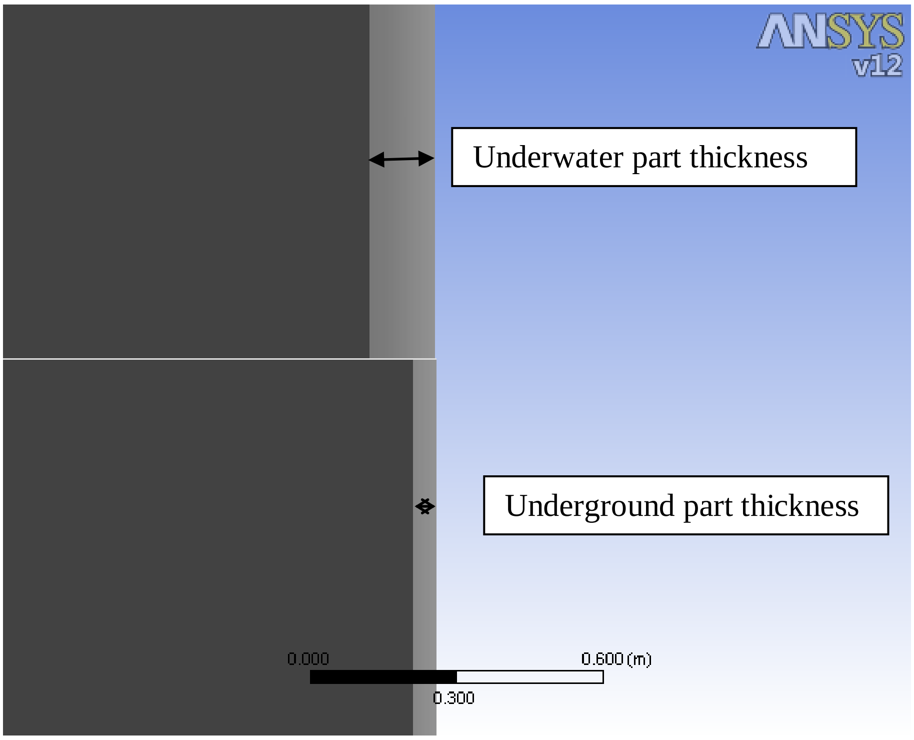

Figure 25: Underwater part and underground part thicknesses

In this case the initial conditions of the monopole structure are:

Volume	202.82 m^3
Total deformation by 1433153,532 N wind force	0.90 m
Total deformation by 800000 N wind force	0.60 m
First natural frequencies	0.281-0.281 Hz

Table 17: Initial conditions of the monopole structure

Taking into account these conditions we make the optimization according two different methods which are SCREENING method and MOGA/NLPQL method.

We will perform the optimization using these two methods because they are the methods available in the ANSYS finite element program. It has also considered the possibility of carrying out the analysis with both methods because, although the MOGA/NLPQL method is more accurate, it can sometimes give wrong solutions depending on the starting point chosen and therefore has decided to take also consider the screening method. This method always leads to the global solution but with less accuracy.

In both cases we will consider the following ranges of variation of thickness for optimization.

Thickness	Initial value (m)	Upper value (m)	Lower value (m)
Tower and transition piece	0.1	0.11	0.09
Underwater part	0.135	0.155	0.115
Underground part	0.05	0.055	0.045

Table 18: Variation ranges of the variables in the optimization process

4.2.2. Screening method

Basing on the above in relation to initial conditions and the ranges of variation of the variables, we perform the optimization according to the screening method, which is the method selected by default in Ansys finite element program.

Can be seen in the following table the optimized values of the structure according to this optimization method.

Thickness	Value (m)
Tower and transition piece	0.091
Underwater part	0.137
Underground part	0.054

Table 19: Optimized thicknesses of the structure by screening method

Because of these values for the three different thicknesses we obtain the following results in the last load test.

Volume	194.03 m^3
Total deformation by 1433153,532 N wind force	0.8832 m
Total deformation by 800000 N wind force	0.578 m
First natural frequencies	0.289-0.289 Hz

Table 20: Optimized solution of the last load test by screening method

Analyzing these results, we can say that the design has improved, mainly because the volume has been reduced significantly, achieving results on the deformation similar to those that were primarily, but also have improved slightly.

We must also comment that the first natural frequencies have been kept within the limits are (0.222 Hz to 0.311 Hz) and that are determined by the specifications defined in Chapter 1 [18].

4.2.3. MOGA/NLPQL method

As explained in the first section of this chapter, this optimization process is a combination of MOGA method and NLPQL method. This combination is carried out such that, first apply the MOGA method that makes an approach to the overall solution and then apply the method NLPQL. NLPQL method is a first-order method based on quasi-Newton methods, to conduct a refinement of the solution reaching a more accurate value.

According to this method the optimized solution obtained is:

Thickness	**Value (m)**
Tower and transition piece	0.09
Underwater part	0.14
Underground part	0.055

Table 21: Optimized thicknesses of the structure by MOGA/NLPQL method

Taking into account the new optimized solution has been achieved, the result obtained in the last load test is:

Volume	194 m^3
Total deformation by 1433153,532 N wind force	0.8796 m
Total deformation by 800000 N wind force	0.575 m
First natural frequencies	0.291-0.291 Hz

Table 22: Optimized solution of the last load test by MOGA/NLPQL method

In this case we note that the result is enhanced still more, because the volume has decreased a little bit more than with the screening method and have achieved also little bit better results for the deformations, while maintaining as in the previous case the first natural frequency within the specifications set out in Chapter 1 [18].

However, although in both cases of optimization we have obtained good results, we notice that the values obtained for the variables in both cases approach the defined boundary limit values.

For this reason it was decided to perform a new optimization taking into account greater ranges of variation for variables.

These new ranges of variation considered are:

Thickness	Initial value (m)	Upper value (m)	Lower value (m)
Tower and transition piece	0.1	0.11	0.05
Underwater part	0.135	0.805	0.115
Underground part	0.05	0.1	0.03

Table 23: Variation ranges of the variables in the second optimization process

4.2.4. Second Screening method

In this case for optimization, we base on the design initial conditions mentioned above and on new ranges of variation considered for the variables.

Can be seen in the following table the optimized values of the structure according to this optimization method.

Thickness	**Value (m)**
Tower and transition piece	0.0769
Underwater part	0.123
Underground part	0.075

Table 24: Optimized thicknesses of the structure by second screening method

Because of these values for the three different thicknesses we obtain the following results in the last load test.

Volume	177.9 m^3
Total deformation by 1433153,532 N wind force	0.856 m
Total deformation by 800000 N wind force	0.55 m
First natural frequencies	0.303-0.303 Hz

Table 25: Optimized solution of the last load test by second screening method

Can be seen this results in the following figures.

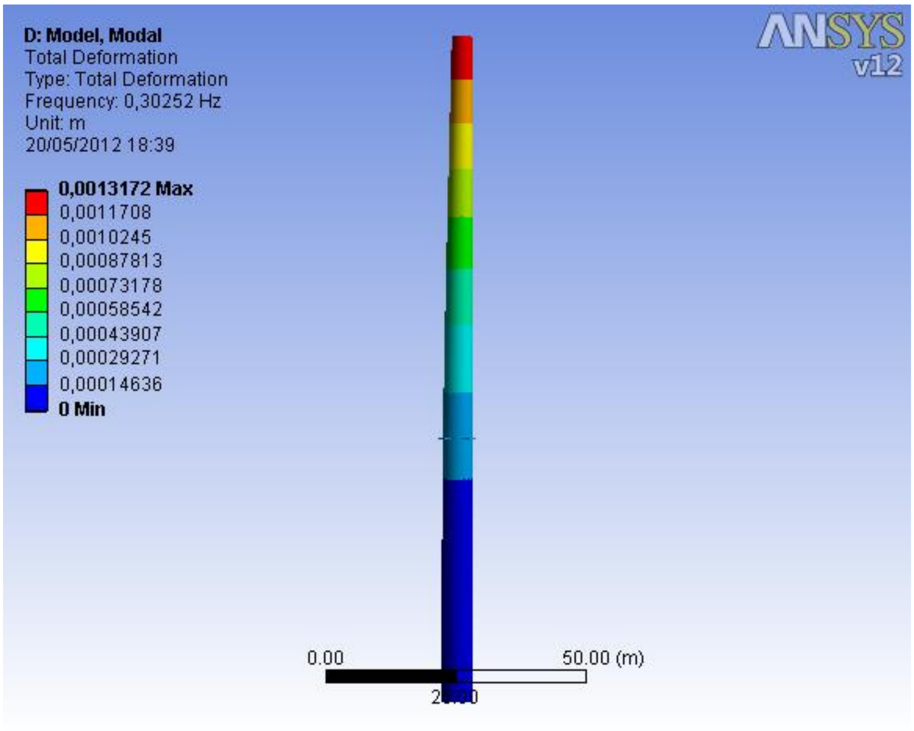

Figure 26: Mode 1 of vibration of the structure by second screening method at 0.30252 Hz

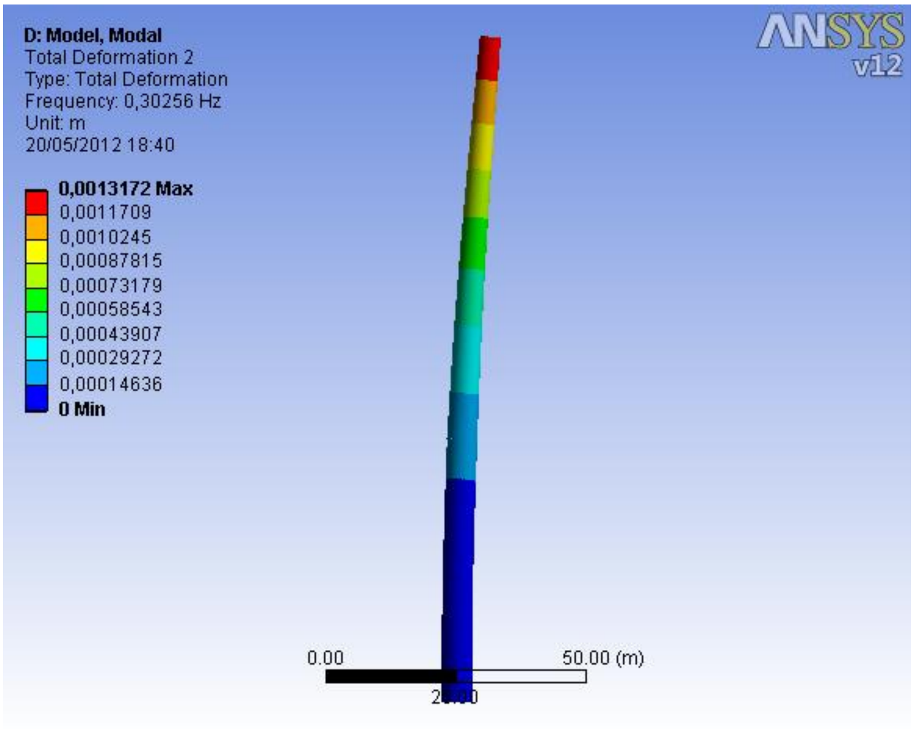

Figure 27: Mode 2 of vibration of the structure by second screening method at 0.30256 Hz

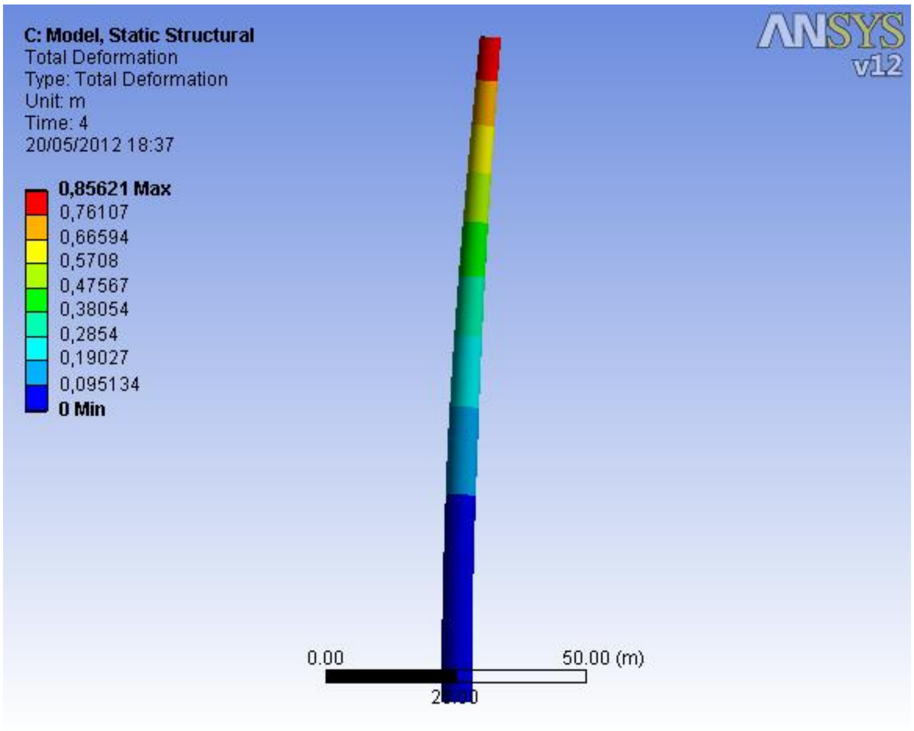

Figure 28: Total deformation of the structure according to maximum wind force of 1433153.532 N by second screening method

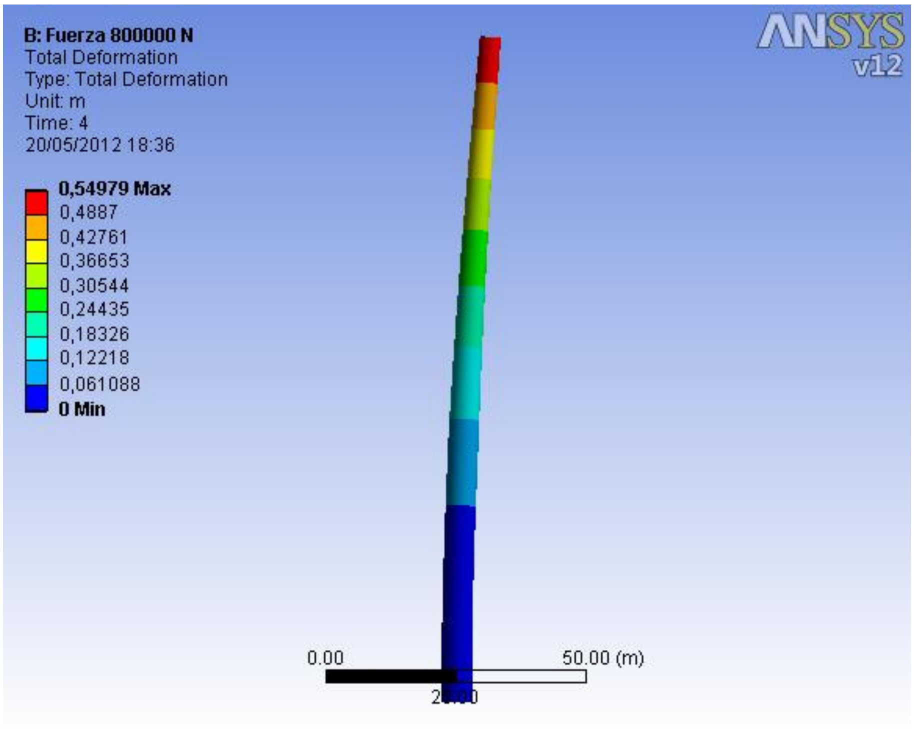

Figure 29: Total deformation of the structure according to maximum wind force of 800000 N by second screening method

Analyzing these results, we found that the design has improved significantly, mainly because the volume has been reduced significantly and also in this case the total deformation suffered during last load test have been reduced considerably. We must also comment that the first natural frequencies have been kept within the limits are (0.222 Hz to 0.311 Hz) and that are determined by the specifications defined in Chapter 1 [18].

4.2.5. Second MOGA/NLPQL method

According to this method the optimized solution obtained is:

Thickness	Value (m)
Tower and transition piece	0.074
Underwater part	0.115
Underground part	0.09

Table 26: Optimized thicknesses of the structure by second MOGA/NLPQL method

Taking into account the new optimized solution has been achieved, the result obtained in the last load test is:

Volume	177 m^3
Total deformation by 1433153,532 N wind force	0.836 m
Total deformation by 800000 N wind force	0.533 m
First natural frequencies	0.307-0.307 Hz

Table 27: Optimized solution of the last load test by second MOGA/NLPQL method

Can be seen these results in the following figures.

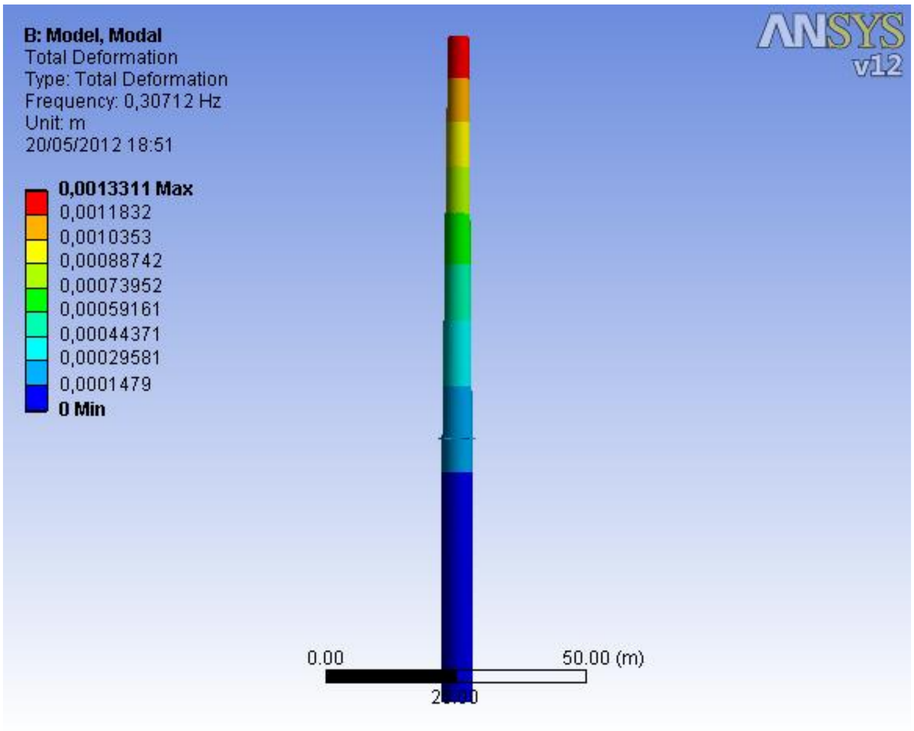

Figure 30: Mode 1 of vibration of the structure by second MOGA/NLPQL method at 0.30712 Hz

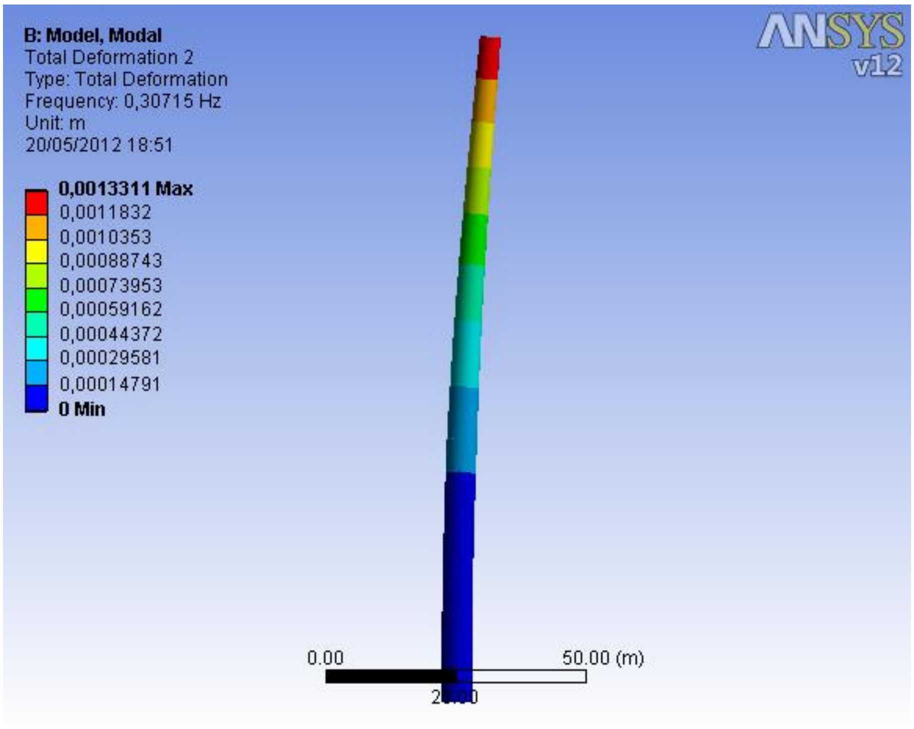

Figure 31: Mode 2 of vibration of the structure by second MOGA/NLPQL method at 0.30715 Hz

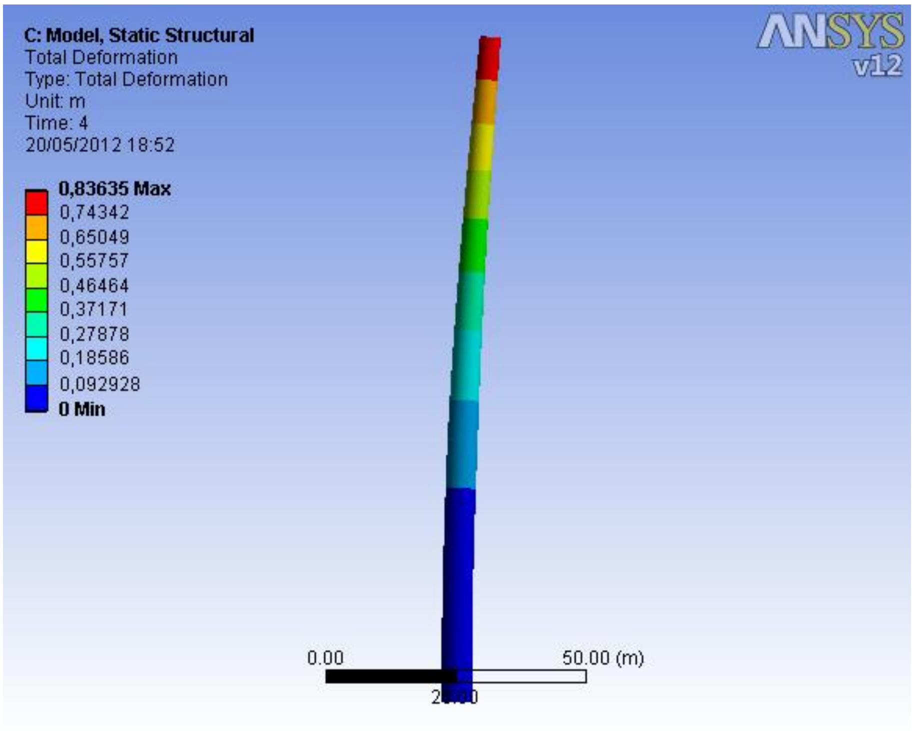

Figure 32: Total deformation of the structure according to maximum wind force of 1433153.532 N by second MOGA/NLPQL method

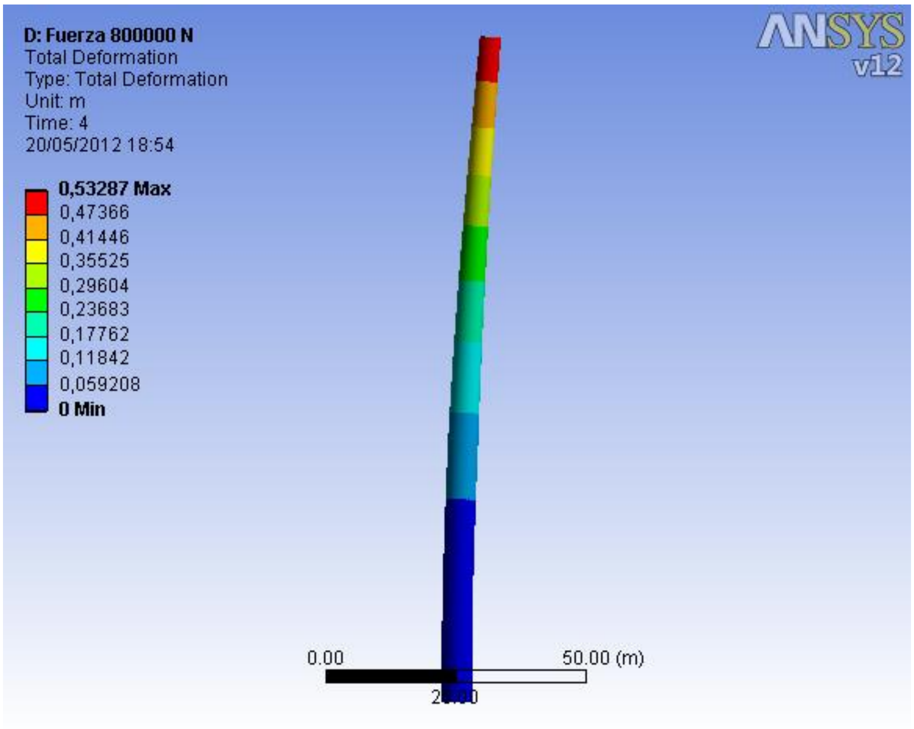

Figure 33: Total deformation of the structure according to maximum wind force of 800000 N by second MOGA/NLPQL method

In this case we note that the result is enhanced still more, because the volume has decreased more than with the screening method and have achieved better results for the deformations, while keeping as in the previous case the first natural frequency within the specifications set out in Chapter 1 [18].

This is because, as mentioned above, the MOGA/NLPQL optimization method is more accurate than the screening method. The main reason is that the MOGA/NLPQL optimization method is a first-order method that uses a lot of information for optimization, while the method is a method SCREENING non-iterative approach with less capable of resolution.

4.3. ANALYSIS OF RESULTS

In this analysis of the obtained results, we will focus on the values for the optimization process carried out with the second MOGA / NLPQL method, because it has been shown that with this method we have obtained the best results when optimizing the wind turbine design.

According to this, it has been demonstrated that the optimization has greatly improved the design had been done previously. We say this because we have gone from a design with a total volume of 202.82 m3 to another with 177 m3, so checking the material needed for our design has decreased significantly, more than 12 %. Compared with the necessary material in the design of jacket-type wind turbine [2] which was also of 202.29 m3, we note that the design has improved as was the case with our first monopile-type design.

In addition to decreasing the volume discussed above, we can analyze the changes in reference to the deformation undergone by the structure in the last load test. We note that the last load test according to a wind load 1433153.532 N, the total deformation changed from 0.91 m to 0.836 m and therefore has been reduced by approximately 8%. We can say therefore that the improvement has been quite considerable.

Analyzing the results of the last load test according to a maximum wind force of 800000 N, we note that the deformation has gone from 0.6 m to 0.53 m, so we see that the reduction has been quite large, more than 10%. Comparing the results we have obtained after this optimization with the results obtained by the design of jacket-type wind turbine, which had a deformation of 0.61 m, can be seen that the improvement on this design is also very good.

We should also note that the first natural frequencies obtained after this optimization are 0.30712 Hz -0.30715 Hz. Therefore taking into account that the limits established by the specifications defined in Chapter 1 [18] which are 0.222 Hz to 0.311 Hz, we find that the design satisfies the required specifications.

In summary it can be said that has improved the design had been done in the first instance, because they have improved all the parameters for the ultimate load test.

5. CONCLUSIONS OF THE THESIS

After the completion of this thesis we can draw some conclusions on all points made.

First, we must comment that this is a thesis with a well defined set of difficulties in terms of the theory refers. This can be seen mainly in the number of references have been necessary to construct a theory that allow us make analysis.

Analyzing numerous articles and publications, we found that in the case of the definition of the model, the model that resembles more reliably the behavior of soil is the distributed spring model. But after many hours of searching and quantity of articles analyzed, it has not found a predetermined method, capable of modeling the soil as a series of distributed springs that allows us to make a more real analysis and therefore to provide a more reliable result.

However, it has been found a theory which allows through the soil characteristics, modeling it as a spring. By combining this theory with that provided by the Ansys finite element program, we were able to create a modeling of soil, as a series of distributed springs. Comparing the theory that we will use with the argument used by the thesis of the jacket-type wind turbine, which considers the soil-structure interaction as fixed, we can say that our assay can be considered more real and therefore provide greater reliability to the design made.

Second, also we have had difficulties to find theories that would allow us to apply for the wind and sea forces during the last load test which is that we have done. Finally, we found a theory that allowed us to apply these forces through a series of calculations and concluded that forces applied were two, one on the highest point of the structure, corresponding to the wind and one at sea level, corresponding to the sea.

Comparing the forces corresponding to the wind as the worst in the last load test, between the thesis of jacket type offshore wind turbine and ours, we observed a clear difference. According to this thesis, the most unfavorable load of 800000 N wind, while the theory we consider the worst case load is about 1.5 MN. Because of this we performed two different ultimate load tests.

Analyzing the results of the first design made and compared with those for the thesis of jacket type offshore wind turbine, we observed that the results were obtained were very similar in all respects.

As to the total volume used in our case is a total volume of 202.82 m^3 while in the case of jacket type offshore wind turbine is 202.26 m^3 [2].

Regarding the deformation undergone in the last load test according to a wind load of 800000 N, our first design suffers a deformation localized in the upper point of the structure corresponding to 0.6 m and the deformation suffered by the jacket-type offshore wind turbine is 0.61 m [2].

Also in regard to the first natural frequency see that both designs have values very similar, although slightly lower in our case, as for our first design are 0.28093 Hz - 0.28095 Hz and the values for the first natural frequencies of the jacket-type offshore wind turbine are 0.2985 Hz -0.3088 Hz [2].

Thus we see that our first design is very similar to the jacket-type offshore wind turbine, as to the results given during modal analysis of frequencies and because of the excitations or forces applied during the last load test

However, we remark that although both designs have similar responses and with virtually the same amount of material used, they have completely different designs, mainly due to the type of foundation that defines them. Comparing both types of foundations can be said that our design adds value, due to the greater easily of construction, because it is a much simpler geometry based on a single tube and also to the greater easily of installation.

Furthermore, it should mention that when performing the analysis were taken into account a greater number of variables, due to the consideration of the soil-structure interaction as a series of distributed springs, while in the case of jacket type offshore wind turbine the soil structure interaction is considered as fixed, they bring to our analysis more realistic and more reliable results.

Once this first design that improves the jacket-type wind turbine design, especially by its greater realism in the analysis and its simplicity, it was decided to design improvement through performing an optimization with the ANSYS finite element program has provided very good results.

As for the material used, it has been achieved a very significant reduction, since it has gone from a total volume of 202.82 m^3 to a volume of 177 m^3, so more than 12 %.

Regarding the deformation undergone in the last load test according to a wind load of about 1.5 MN, it has been changed from 0.91 m to 0.836 m, approximately an 8 %, and because of a maximum wind load of 800000 N the deformation has changed from 0.60 m to 0.53 m, approximately a 10 %. So we can affirm that in this aspect improvements have been also very considerable.

We should note that during the optimization performed, it has managed to maintain the first natural frequencies within the limits set by the specifications discussed in Chapter 1 [18], since these limits are from 0.222 Hz to 0.311 Hz and the values were obtained are 0.30712 Hz and 0.30715 Hz.

Finally we can conclude that through this thesis has achieved the goal proposed, which was create, analyze and optimize the design of a monopile type offshore wind turbine based on soil-structure interaction defined with distributed springs model. We can say this, as has been improved by designing a monopile-type wind turbine the design of a jacket-type wind turbine and then improved this first design in a very remarkable manner.

It should also be aware that all these analysis, optimizations and so on, have been carried out considering a larger number of variables, due to the modeling of soil-structure interaction through a distributed system of springs, which provides more realistic and reliable design, so it is considered a more complex model than the corresponding jacket-type offshore wind turbine

6. GENERAL CONCLUSIONS

During the completion of the thesis I have drawn a number of very important conclusions, mainly concerning the personal and the professional field.

As for the professional field has helped me develop good concepts about wind energy, more particularly, offshore wind energy that I previously unknown and that I think can be useful, because renewable energy will be important in the near future. Also within these renewable energies, wind energy is one of the branches that have more future, which is developing more and therefore may provide a good chance of work.

Apart from the possibilities offered by working, I have also noted that I have discovered an area of work that I previously unknown and I was pleasantly surprised. I found a very interesting field for all the possibilities offered, by the ability of development that has, for the ability to innovate, etc... and therefore the number of possibilities for carrying out interesting and innovative projects, that enable the acquisition and discovery new knowledge that allow us to develop both as persons and as engineers.

In the professional field, is also remarkable, that not only have acquired a number of theoretical knowledge, it has also been a project that has allowed me to implement all knowledge that make it much more interesting work done. It was very interesting for me to implement this knowledge by using the Ansys finite element program.

As for the difficulties encountered during the implementation of the thesis, can been summarized in two mainly. The first is the great difficulty of finding theoretical information useful for the realization of this thesis, because it seems to be that offshore wind energy is not yet a highly developed industry and access to information is difficult. The second difficulty has been found using the finite element program Ansys. This is because at the beginning of the realization of the thesis, I not had any prior knowledge about this software and therefore the learning carried out while the work is done for the thesis has generated many problems.

In my opinion I think that could be carried out several subsequent works, taking as its starting point the work.

First consider one of the fields that most development could have, would be appropriate to model the soil and therefore the improvement of theory considered for modeling the ground as a series of springs distributed. This consideration is mainly because it has been in this area in which greater lack of information has been found and is therefore well suited to a field study.

Secondly I think another area suitable for future thesis is the practical application of theoretical concepts developed. This field is also possible to develop because it has been another area where most problems have been obtained, possibly due to ignorance of the possibilities offered by the software used or possibly due to deficiencies offering the program.

Once carried out work in these fields, the study could be extended to new offshore wind turbine models that provide better answers, by the discoveries have been made and by other types of tests that allow a more complete study.

These would be in my opinion the major fields of study that could be considered for further development of offshore wind energy.

On a personal level I must emphasize that it has been helped me to acquire a number of very important skills in all aspects. For example, I learned to organize myself to perform complex tasks and lasting in the time required for a period of reflection prior to face work with guarantees. It also helped me to learn to solve problems that have arisen, through research and subsequent analysis of the information obtained.

One of the aspects that should be highlighted in this personal level, is that I have learned to search information more efficiently, to select the information and interpret it properly. I think this is one of the most remarkable aspects, because it is essential in any job, besides being necessary in many personal situations that can affect anyone.

Finally, I believe it's been a very rewarding experience because it allowed me to develop as a person and as an engineer thanks to all the skills, abilities and knowledge that has been achieved in all aspects.

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